Part IV

Seismic Evaluation Procedures Developed Uniquely for the DOE

10. EQUIPMENT CLASS EVALUATIONS USING SCREENING PROCEDURES OR GENERAL GUIDELINES

Chapter 10 contains a summary of equipment class descriptions and parameters based on earthquake experience data, test data, and analytical derivations. The classes of equipment contained in Chapter 10 are not from the SQUG GIP (Ref. 1). Much of the information in Chapter 10 is from DOE references. Table 2.1-4 lists the principal references and authors for the sections in Chapter 10. An item of equipment must have the same general characteristics as the equipment in the screening procedures and general guidelines. The intent of this rule is to preclude items of equipment with unusual designs and characteristics that have not demonstrated seismic adequacy in earthquakes or tests.

The screening procedures in Sections 10.1.1, 10.4.1, and 10.5.1, for evaluating the seismic adequacy of piping, HVAC ducts, and unreinforced masonry (URM) walls respectively, cover those features which experience has shown can be vulnerable to seismic loading. These procedures are a step-by-step process through which the important equipment parameters and dimensions are determined, seismic performance concerns are evaluated, the equipment capacity is determined, and the equipment capacity is compared to the seismic demand. Sections 10.1.1 and 10.4.1 have been technically reviewed and used extensively at several DOE sites including Savannah River Site and Rocky Flats Environmental Technology Center.

The general guidelines for evaluating the seismic adequacy of the equipment classes in the other sections of Chapter 10 cover those features which experience has shown can be vulnerable to seismic loading. The sections contain practical guidelines and reference to documents that can be used to implement an equipment strengthening and upgrading program. The relatively simple seismic upgrades are designed to provide cost-effective methods of enhancing the seismic safety of the equipment classes in Chapter 10. Sections 10.3.1 and 10.1.2 summarize information from portions of a DOE document that has undergone extensive technical review. Sections 10.2.1, 10.2.2, 10.2.3, 10.3.2, 10.5.2, and 10.5.3, on the other hand, are based on walkdown and seismic strengthening efforts at several DOE sites including Los Alamos National Laboratory and Lawrence Livermore National Laboratory.

10.1 PIPING SYSTEMS

10.1.1 PIPING

This section is the "Procedure for the Seismic Evaluation of Piping Systems Using Screening Criteria", WSRC-TR-94-0343 (Ref. 59) which was developed by the Westinghouse Savannah River Company. Some of the background material for this section is contained in References 52 through 55 and the technical review of this section is summarized in Reference 27.

10.1.1.1 Objective

This procedure may be used to evaluate the seismic adequacy of piping systems within the Scope, Section 10.1.1.2, and subject to the Cautions, Section 10.1.1.3.

The procedure may be used alone or with the rest of the DOE Seismic Evaluation Procedure, depending on the piping system's required function, listed in Table 10.1.1-1.

Table 10.1.1-1 Procedures Applicable to Required Piping System Functions

FUNCTIONS	Delivers Flow?	Equipment Operating?	Leak Tight?	Not Fall?	PROCEDURE
Operability	Yes	Yes	Yes	Yes	Piping Screens and DOE Seismic Evaluation Procedure for Equipment
Maintain Integrity of Pressure Boundary	No	No	Yes	No	Piping Screens and DOE Seismic Evaluation Procedure for Equipment Anchorage
Position Retention	No	No	No	Yes	Subset of Piping Screens

Features of a piping system that do not meet the screening criteria are called outliers. Outliers must be resolved through further evaluations (see Chapter 12), or be considered a potential source of seismically induced failure. Outlier evaluations, which do not necessarily require the qualification of a complete piping system by stress analysis, may be based on one or more of the following: simple calculations of pipe spans, search of the test or experience data, vendor data, industry practice, or other appropriate methodology.

10.1.1.2 Scope

This procedure applies to existing (installed), safety or non-safety related, above ground metallic piping or tubing systems constructed of materials listed in ASME B31.1 (Ref. 90), ASME B31.3 (Ref. 91), NFPA (Ref. 92), or AWWA (Ref. 93), with the following restrictions:

1. Pipe materials must be ductile at service temperatures. Cast iron materials are excluded. Non ferrous alloys with a specified ultimate tensile strength (UTS) of less than 30 ksi are excluded. Welded aluminum materials are excluded. Soldered joints are outliers.

2. Diameter-to-thickness ratio (D/t) of pipe must be 50 or less. In terms of pipe thickness (t), the thickness must be greater than the diameter (D) divided by 50.

3. Operating temperature must be below 250°F, but above -20°F.

4. The facility's Seismic Demand Spectrum (SDS) must meet the requirements of Chapter 5.

Commentary

1. While the focus of seismic experience has been mostly on welded steel piping, there is no evidence that welded piping constructed of metals other than gray cast iron has performed poorly in past earthquakes. Test and earthquake experience of piping systems is contained in References 94 through 99.

   Except for aluminum, non ferrous pipe materials allowed by the ASME B31.3 (Ref. 91) code have UTS of 30 ksi or better. Welded aluminum is excluded since many grades of aluminum alloy have low specified ultimate and yield strengths, and tend to have low fatigue strength and limited ductility in the heat affected zone.

   The screens may be used for copper piping. The UTS of weldable grades of copper and bronze piping exceeds 30 ksi. Copper tubing and piping can also be brazed, and a properly brazed joint is stronger than the pipe.

   Soldered joints operating at ambient or higher temperatures exhibit, with time, a reduced strength. At cryogenic temperatures they tend to become brittle. Soldered joints, unlike brazed joints, must be considered outliers.

   Pipe materials must be ductile at service temperatures, having total elongation at rupture greater than 10%. Table 10.1.1-2 shows such properties for common piping materials at room temperature. When judging material ductility, the review team must consider the effect of material degradation on these properties, particularly the potential for reduced elongation caused by lowered ductility.

   Cast iron or brittle elements in a ductile piping system are outliers, but they may be accepted (by other appropriate procedures) if proven to be located in low seismic stress areas, and not susceptible to impact.

   Seismic induced deflection or loads at groove type mechanical joints shall be limited to vendor listed allowables or test based limits.

   Dynamic seismic testing of threaded joint pipe sections indicates that they are prone to leakage under large rotations. For threaded joints, the span between lateral supports, in Section 10.1.1.10, have been reduced accordingly.

2. The seismic testing and earthquake experience data is mostly from standard or thick wall pipe. The screening criteria apply directly to piping systems with a D/t ratio of 50 or less.

3. Below 250°F, thermal expansion loads are small for the purpose of seismic evaluation. The review team should identify unusually stiff piping configurations where the 250°F rule is questionable. Materials lose ductility at low temperatures. Therefore, piping operating below -20°F are considered outliers.

4. Limiting the screening criteria to the specified free field horizontal spectral acceleration is a precaution introduced to remain within the scope of earthquake experience data for equipment.

Table 10.1.1-2 Typical Properties of Common B31.3 Piping, Tubing, Fitting, and Support Members Materials at Room Temperature

DESCRIPTION	MATERIAL	BASIC ALLOWABLE (ksi)	YIELD STRENGTH (ksi)	ULTIMATE STRENGTH (ksi)	ELONGATION IN 2" DIA. ROUND SPECI. (min. %)
Structural Steel	A36	17.8	36.0	58.0 - 80.0	20 - 23
Carbon Steel Pipe	A53, GR. B	20.0	35.0	60.0	22 - 23
Carbon Steel (Forged Fitt.)	1A105, FR. CL-70	23.3	36.0	70.0	18 - 30
Carbon Steel (Seamless Pipe)	A106, GR. B	20.0	35.0	60.0	16 - 30
Pipe Fitting	A234 GR. WPB	20.0	35.0	60.0	14 - 30
Carbon Steel Bolt	A307, GR. B	13.7	36.0	60.0 - 100.0	18
Stainless Steel Pipe	A312, GR. TP-304L	16.7	25.0	70.0	25 - 35
Copper Tube	Various types	6.0 - 15.0	9.0 - 40.0	30.0 - 50.0	25
Red Brass Pipe	B43 Temp. 061	8.0	12.0	40.0	35

10.1.1.3 <u>Cautions</u>

1. The screening criteria are not meant to be a design tool. The applicable code should be used at the design and layout stage. The screening criteria are not equivalent to compliance with the seismic design requirements of ASME B31.1 (Ref. 90), ASME B31.3 (Ref. 91), ASME

Boiler and Pressure Vessel Code Section III (Ref. 100), NFPA-13 (Ref. 92), AWWA (Ref. 93), AISC (Ref. 81), or AISI (Ref. 101). An existing piping system may comply with the screening criteria but not with the design codes' seismic requirements, and vice-versa.

If a piping system has been designed and constructed to comply with the seismic design provisions of a reference code, it is not necessary to evaluate its seismic adequacy using this procedure. However, the review team may chose to address the provisions of screens 10.1.1.7 "Internal Degradation", 10.1.1.8 "External Corrosion" and 10.1.1.18 "Interaction with other structures" of this procedure, since these considerations are not typically addressed in design codes.

If seismic loads were not included in the original code design of the piping system, the review team may evaluate the seismic adequacy of the non-seismically installed piping system using this procedure, with approval from the owner and/or jurisdiction as appropriate. As an alternative, the review team may evaluate the seismic adequacy of the installed system using the seismic design provisions of the reference code.

2. Application of the screening criteria must reflect the consensus of a seismic review team of two or more degreed engineers, each engineer having the following qualifications (see Section 3.2.2):

   a. a minimum of five years experience in seismic design and qualification of piping systems and support structures

   b. capability to apply sound engineering judgment, based on the knowledge of the behavior of piping systems in actual earthquakes and seismic tests.

3. Qualified users of the screening criteria must complete a training course (see Section 3.2.2) and successfully pass an examination (as appropriate) in the following topics:

   a. content and intent of the screening criteria

   b. piping and pipe support design requirements of ASME B31.1 (Ref. 90), ASME B31.3 (Ref. 91), NFPA-13 (Ref. 92), AWWA (Ref. 93), AISC (Ref. 81), and AISI (Ref. 101)

   c. piping and pipe hanger standards

   d. piping materials and degradation mechanisms

   e. support anchorage rules of the DOE Seismic Evaluation Procedure

   f. earthquake and seismic test experience data for piping systems

4. The screening criteria rely on the considerable body of piping test, earthquake data and analytical design practice to screen and identify the following key attributes which may lead to seismically induced failures of piping systems:

   a. <u>Material condition</u>: Poor construction details and material degradation are at the source of many seismic failures observed in piping systems. Construction quality and material condition are thoroughly covered in the screens.

b. Anchor motion: Excessive anchor motion propagated through equipment and headers has resulted in seismic failures of piping systems. The screens provide for protection against excessive anchor motion.

c. Brittle features: Brittle materials and certain fittings and joints are screened out to avoid non-ductile piping systems.

d. Interactions: Experience data shows several failures traceable to seismic interactions on the piping systems the potential for interactions. Screens are provided to assess the potential for credible and significant interactions.

10.1.1.4 Documentation

The review team shall complete a Piping Seismic Evaluation Work Sheet (SEWS 10.1.1 in Chapter 13) for each piping system. Similar piping systems may be documented in a single SEWS 10.1.1.

The technical basis for judging each screening criterion shall be described on attached sheets and cross referenced in the corresponding notes column of the SEWS 10.1.1.

Written calculations shall be sufficiently detailed to clarify the purpose of the calculation and the conclusion. All assumptions shall be noted.

The method and calculations to resolve outliers shall be documented.

The purpose of each screening criterion is included in this procedure and explained in the required training course.

For each piping system, a complete documentation package will be assembled consisting of the P-SEWS with attached notes and calculations, sketches, and photographs.

Documentation should be sufficient for independent review by an experienced piping engineer trained in the application of this procedure.

10.1.1.5 Required Input

1. Piping System ID

   Record the appropriate piping identification numbers, such as line numbers, chronological numbers, calculation numbers, equipment list item numbers, etc.

2. System Description and Fluid Boundaries

   Piping system descriptions such as system, subsystem, or line number must clearly communicate the scope of the seismic review (boundary points) on a flow diagram sketch. All branch lines shall be identified, and seismic/non-seismic fluid boundaries shall be noted.

3. Piping System Function and Contents

   The contents and function of the piping system during and after the earthquake must be described and categorized as operability, integrity of pressure boundary or position retention (refer to Table 10.1.1-1). For operability, identify active equipment.

4. Piping Layout and Structural Boundaries

Isometric sketches, based on visual inspection, must be sufficient for piping engineers to visualize system response and calculate approximate span equivalent lengths.

Structural boundaries, along with support types and locations shall be noted. If adjacent walls or structures are relied on for seismic restraint, these features shall also be noted. In-line equipment and concentrated masses shall be noted where they contribute to significant weight.

5. Piping System Location and Reference Drawings

Record the piping system location, such as building, floor or room number.

If the piping system spans different buildings or floors, note all locations.

A list of reference drawing numbers and revisions used in the evaluation, such as flow diagrams, piping arrangement diagrams, isometrics, equipment drawings, etc. is required. A separate sheet may be used if needed.

6. Piping Materials and Sizes

List all pipe materials, sizes (nominal pipe size and schedule or thickness) and the references used to determine this information (such as specifications or drawings).

7. Weights

Linear weight (lb/ft) of piping and contents must be recorded for each size of pipe. Noted contents (liquid, gas, air, steam, etc.) must be the same as expected during a postulated earthquake.

Note the linear weight (lb/ft) of insulation and the references used to determine this information (such as specifications or drawings). Record weight of in-line components and eccentricities, as necessary.

8. Concurrent Pressure and Temperature

Specify the pressure and temperature conditions expected concurrent with the postulated earthquake. The pressure values will be used in the component rating screen (refer to Construction Quality). The temperature must be below 300°F for the screens to apply (refer to Applicability Section).

9. Input Response Spectra (see Section 5.2)

The input response spectra are used in several screens and may be necessary for the resolution of outliers.

The review team shall document the appropriate ground and/or floor response spectra, applicable references, and status (final or preliminary). Final response spectra are required to finalize the evaluation.

The ground response spectra (at 5% damping) shall be used for piping supported from grade. (see Section 5.2)

The floor (in-structure) response spectra (at 5% damping) shall be used for piping supported above grade. (see Section 5.2)

If the piping terminal ends are at large flexible equipment, seismic anchor motion of the equipment nozzles shall be considered.

If the piping spans between buildings, the relative anchor motions shall be considered. Relative building movements shall be obtained from the building structural analysis.

10. Applicability

Limits and conditions as given in the Applicability section must be met, to ensure that the material, size (D/t), temperature (250°F and -20°F) and input acceleration of evaluated piping is appropriate for this screening procedure.

10.1.1.6 Construction Quality (Screen 1)

Screen 1 - Piping, components and supports shall be undamaged and of good construction.

Commentary

An assessment shall be conducted of the design, welding, and fabrication quality, as well as all visible damage to the piping and the supports, prior to applying the screening criteria.

The piping system must have been fabricated and examined in accordance with ASME B31.1 (Ref. 90), ASME B31.3 (Ref. 91), AWWA (Ref. 93), or NFPA (Ref. 92).

Pressure ratings for branch connections and fittings shall be checked for adequacy. Systems with pressures in excess of that allowed for ANSI B16.5 (Ref. 102) class 2500 are considered outliers.

Standard pipe fittings manufactured to specifications must have the same pressure rating as their corresponding size and schedule of straight pipe. Unreinforced branch connections, or pipe fittings or couplings unlisted in the applicable standards, or which lack stated pressure ratings, could have significantly lower pressure ratings and seismic capability than their complementary straight pipes, in which case they are outliers.

The piping and supports shall be visually inspected for adequate quality of design, fabrication, installation and maintenance. Instances of poor quality shall be noted. Where piping is not accessible for direct visual examination (covered with insulation, located in inaccessible areas, etc.), construction quality may be based on as-built construction and maintenance records confirmed to be up-to-date.

Signs of poor construction quality or subsequent damage include:

1. excessive distortion of piping or supports

2. brazed joints, apparently of good quality, but without a thin layer of brazing or solder visible where the tube extends beyond the fitting socket

3. uneven, undersized or damaged welds

4. unusual or temporary repairs

5. evidence of interference having caused significant bearing, scratch marks or distortion to the pipe metal or to components

6. a pipe dislodged from its support so that the weight of the pipe is distributed unevenly on the hangers or saddles

7. the deformation of a thin vessel wall in the vicinity of a pipe attachment

8. pipe supports forced out of position by expansion or contraction of the piping

9. the shifting of a base plate, breaking of a foundation, or shearing of foundation bolts of mechanical equipment to which piping is attached

10. missing nuts or bolts

11. signs of leakage (discoloration, dripping, wet surface)

12. cracks in connecting flanges or the cases of pumps or turbines to which piping is attached

13. deterioration of protective coatings, fireproofing or other periodic maintenance conditions

14. general physical damage

15. movement or deterioration of concrete footings

16. failure or loosening of foundation bolts

17. insecure attachment of brackets and beams to the support

18. restricted operation of pipe rollers or slide plates

19. insecure attachment or improper adjustment of pipe hangers

20. broken or defective pipe supports

21. oversized bolt holes

10.1.1.7 Internal Degradation (Screen 2)

Screen 2 - Piping and components shall be free of significant internal degradation.

Commentary

Significant degradation refers to that which may affect the pressure integrity of the piping system. The potential for internal degradation must be investigated and documented from two aspects.

1. the piping system operating performance records, and

2. a metallurgical assessment

It is unnecessary to perform new nondestructive surface or volumetric examinations of the piping system for this screen. The review of performance records and metallurgical assessments are to be based on existing data. If either source of information is unavailable or suggests potential internal degradation, the system must be classified as an outlier.

If the condition of the piping system is judged adequate, but some degradation is expected to occur in the future, the system must be subjected to periodic in service inspection or evaluated for the effects of the expected degradation.

10.1.1.7.1 Operating Performance Record

The system cognizant engineer must identify and assess past maintenance, repairs and replacements performed on the piping system, or on similar systems, to judge if they indicate potential metallurgical or mechanical degradation mechanisms.

The system cognizant engineer must identify any history of abnormal events or loadings, such as flow induced vibration, water hammer, misalignment, binding, and excessive temperature cycling, to judge if they may have caused system degradation due to fatigue or localized yielding.

Evidence of pipe leakage, pipe repair, support failures, or abnormal vibration may indicate significant cyclic loading, which shall be resolved.

10.1.1.7.2 Metallurgical Assessment

The metallurgical assessment of the piping systems must be performed with the help of materials engineering. When considering materials, fluids and operating conditions, the materials engineer must judge the potential for reduced performance capability resulting from material degradation, erosion or corrosion.

10.1.1.7.3 Guidance: Susceptible Areas

The following areas are most susceptible to corrosion, erosion, and other forms of material degradation.

1. points at which condensation or boiling of acids or water is likely to occur
2. points at which acid carryover from process operations is likely to occur
3. points at which naphthenic or other organic acids may be present in the process stream
4. points at which high-sulfur streams at moderate-to-high temperatures exist
5. points at which high- and low-temperature hydrogen attack may occur
6. dead ends subject to turbulence, or where liquid-to-vapor interface or condensation occur
7. valve bodies and trim, fittings, ring grooves and rings, and flange facings
8. welded areas subject to preferential attack
9. catalyst, flue-gas, and slurry piping
10. steam systems where condensation occurs
11. ferrous and nonferrous piping subject to stress corrosion cracking
12. alkali lines subject to caustic embrittlement and resultant cracking

13. areas near flanges or welded attachments that act as cooling fins, causing local corrosion because of temperature differences

14. locations where impingement or changes in fluid velocity can cause local accelerated corrosion or erosion

15. points of accidental contact or insulation breakdown that causes contact of dissimilar metals

16. an area where steam or electric tracing contacts piping handling material such as caustic soda, where concentrated heat can cause corrosion or embrittlement

17. an area immediately downstream of a chemical injection point, where localized corrosion might occur in the reaction zone

18. heat-affected zones (around and in welds) in non-post weld heat-treated carbon steel piping in amine service

19. dissimilar metal welds

20. piping subject to mechanical or flow induced vibration.

The potential for general corrosion or erosion that could result in pipe wall thinning shall be assessed. If wall thinning potential exists in the material or environment, sample measurements shall be taken. If the predicted thinning exceeds 20% of the pipe wall for the planned life of the piping system, the system is an outlier.

If stress corrosion cracking is likely, examinations shall be performed.

The hazard of embrittlement (due to hydrogen, hydrogen cracking, irradiation, thermal aging, etc.) for the planned life of the piping system shall be assessed. If it is possible for pipe ductility (total elongation at rupture) to be reduced by 10% or more, the system is an outlier.

10.1.1.7.4 <u>Guidance: Material Compatibility</u>

The following possible material conditions must be evaluated, along with other service specific conditions:

1. Carbon Steel, and Low and Intermediate Alloy Steels

    a. possible embrittlement when handling alkaline or strong caustic fluids

    b. possible hydrogen damage to piping material when exposed (under certain temperature-pressure conditions) to hydrogen or aqueous acid solution

    c. possible stress corrosion cracking when exposed to wet hydrogen sulfide, and the further possibility of deterioration (sulfidation) in the presence of hydrogen sulfide at elevated temperatures

    d. the need to limit maximum hardness of metals in applications subject to stress corrosion

2. High Alloy (Stainless) Steels

   a. possible stress corrosion cracking of austenitic stainless steels exposed to media such as chlorides and other halides either internally or externally as a result of improper selection or application of thermal insulation

3. Nickel and Nickel Base Alloys

   a. possible stress corrosion cracking of nickel-copper alloy (70Ni-20Cu) in hydrofluoric acid vapor if the alloy is highly stressed or contains residual stress from forming or welding

4. Copper and Copper Alloys

   a. possible dezincification of brass alloys

   b. susceptibility to stress-corrosion cracking of copper-based alloys exposed to fluids such as ammonia or ammonium compounds

   c. possible unstable acetylene formation when exposed to acetylene

10.1.1.8 External Corrosion (Screen 3)

Screen 3 - Piping, components and supports shall be free of significant external corrosion.

Commentary

In reviewing the piping system for signs of corrosion, the seismic evaluation team must consult the materials engineer for questionable conditions.

Significant corrosion refers to metal thickness loss of more than 20%. A surface discoloration or thin layer of rust does not harm structural integrity. Rust forms a surface coating which protects the inner metal from further corrosion.

A loss in thickness can be measured by comparing the pipe diameter at the corroded area with the original pipe diameter. The depth of pits can be determined with a depth gauge.

Stainless steel, copper, nickel, and their alloys are typically used in B31.3 (Ref. 91), and resist atmospheric corrosion. They may be accepted without further review. Iron and carbon (low alloy) steels, however, may be subject to attack, particularly in areas where moisture can accumulate. If piping is insulated and made of iron or carbon/low alloy steel, insulation should be removed at 3 accessible and susceptible points and the pipes inspected for corrosion.

Significant corrosion (uniform loss of more than 20% of metal thickness) can impair the ability of the supports or piping to carry loads. For supports, areas to consider include threaded sections and pipe-clamp or pipe-saddle interfaces. Local metal loss exceeding 20% of the wall thickness may be acceptable, but each occurrence must be evaluated.

10.1.1.8.1 Atmospheric Corrosion

When metals such as iron or steel are exposed to the atmosphere, they will corrode due to the presence of water or oxygen. Below 60% humidity, corrosion of iron and steel is negligible. To prevent atmospheric corrosion, it is necessary to protect the surface of the metal from water by means of a protective barrier or coating.

The normal rate of atmospheric corrosion of unpainted steel in rural atmospheres is low, ranging from 0.001 to 0.007 inches per year. However in some atmospheres, a steel corrosion rate of 0.05 inches per year is possible. The rate of corrosion accelerates at any break in a protective coating because the exposed metal at the break becomes anodic to the remaining metal surface. At such breaks, deep pits will form.

Equipment which is located next to boiler or furnace stacks and exposed to corrosive gases such as sulfur dioxide and sulfur trioxide is subject to accelerated corrosion. These gases, dissolved in water condensate from flue gas, rain, or mist, form dilute acids which act as electrolytes. In addition, chlorides, hydrogen sulfide, cinders, fly ash, and chemical dusts present in industrial atmospheres may act in a similar manner.

10.1.1.8.2 Corrosion Under Insulation and Fireproofing Materials

Inadequate weatherproofing on piping allows moisture to penetrate to the underlying steel, where hidden corrosion takes place. Such hidden corrosion is often severe in refrigeration systems. The skirts of all vessels, regardless of operating temperatures, are subject to severe corrosion under insulation or fireproofing. Cracks in fireproofing concrete, particularly at the top where the concrete ends, also allow moisture to penetrate and hidden corrosion to occur. Protective organic coatings may be useful, especially in seacoast areas where chlorides can come from the air rather than from the insulation. Inhibited insulation, or insulation free of water-soluble chlorides, should be used with austenitic (300 series) stainless steels to prevent stress corrosion cracking.

Defects in protective coatings and the waterproof coating of insulation will permit moisture to contact the piping. When defects are found in the waterproof coating of insulation, enough insulation should be removed to allow the extent and severity of corrosion to be determined. Sections of insulation should be removed from small connections, such as bleed lines and gauge connections, since these locations are particularly vulnerable to atmospheric attack due to the difficulty of sealing the insulation.

10.1.1.8.3 Corrosion of Piping at Contact Points

Piping installed directly on the ground suffers severe corrosion on the underside from dampness. If grass or weeds are allowed to grow beneath and around piping, the underside of the pipe will remain damp for long periods and will corrode. Lines laid directly on supports, or hung by clamps, often show crevice corrosion at the contact points.

Lines that sweat are susceptible to corrosion at support contact points, such as under clamps on suspended lines. Piping mounted on rollers or welded support shoes is subject to moisture accumulation and corrosion. Loss of vapor-sealing mastic from the piping insulation can result in local corrosion. Pipe walls inside open-ended trunnion supports are subject to corrosion. These points should be investigated.

10.1.1.8.4 Corrosion of Structures

Structures that provide crevices where water may enter and remain for long periods are subject to severe corrosion. Examples are structural members placed back to back, and platforms installed close to the tops of towers or drums. Structures located near furnace stacks and cooling towers are particularly susceptible to this type of attack.

10.1.1.8.5 Leakage

The walkdown team must check for the possibility of leaking fluids, suggested by local discoloration or wet surfaces on the pipe or floor.

Bolted joints such as valve packings or flanges may leak. This is especially true for water lines following prolonged periods of sub-freezing weather. Performance records of frozen water pipes show incidents of leakage due to frozen water expanding through and distorting flange gaskets.

Leaks from bolted joints allow fluid to either collect on the pipe or drip onto other systems. In areas where leaks are encountered, the walkdown team should ensure either that the bolts and fluid are compatible or that the bolting has not been subjected to process fluid attack from gasket leakage.

10.1.1.9 Span Between Vertical Supports (Screen 4)

Screen 4 - Piping shall be well supported vertically.

Commentary

A piping system may be considered well supported for deadweight if the equivalent span length between vertical supports, for liquid or gas service, is as shown in Table 10.1.1-3, which lists acceptable vertical support spacing for this screen. The spans in this table correspond to 150% of the ASME B31.1 suggested pipe support spacing provided in Table 121.5. The ASME B31.1 values are based on a bending stress of 2300 psi and a maximum sag of 0.1 inch. Since these values are low, it has been judged reasonable to use 150% of the ASME B31.1 span lengths for installed systems.

Table 10.1.1-3 Equivalent Span Between Vertical Supports

Nominal Pipe Size (in)	Liquid Service (ft)	Gas Service (ft)
1	10	13
2	15	19
3	18	22
4	21	25
6	25	31
8	28	36
12	34	45
16	40	52
24	48	63

The equivalent span length L_{ei} in a given direction i is defined as: $L_{ei} = (W_{pi} + W_{ci}) / W$, [ft]

W_{pi} = Weight of pipe length in span between consecutive supports in direction i, including insulation and contents [lb]

W_{ci} = weight of in line components in span [lb]

W = weight per unit length of pipe size, insulation and contents in span [lb/ft]
The equivalent span length for gas service may be used for evaluating empty, normally dry, pipe spans.

Vertical loading can be resisted by engineered deadweight supports, or structures that are not considered deadweight supports, such as penetrations through walls, certain types of box beam horizontal restraints, and floor slabs.

The following vertical support configurations shall be considered outliers in seismic screening evaluations.

1. friction clamp connections

2. shallow pipe saddle support or pipe rolls

3. bottom support if not positively attached to the pipe and floor, and if the lateral movement of the pipe could possibly tip the support

4. pipe resting on a support, free to slide laterally so as to fall off the support

5. A clamp on a vertical riser without positive attachment to the pipe, such as lugs above the clamp.

10.1.1.10 Span Between Lateral Supports (Screen 5)

Screen 5 - Piping shall be sufficiently restrained in the lateral direction.

Commentary

A piping system may be considered sufficiently restrained in the lateral direction if the equivalent lateral span length for liquid or gas service does not exceed three times the spans in Table 10.1.1-3, which corresponds to 4.5 times the ASME B31.1 (Ref. 90) suggested vertical pipe support spacing. This span is to be divided by 2.3 (stress intensification factor for threaded joints) for pipe sections which contain threaded joints.

The 4.5 times the B31.1 deadweight spans for spacing of lateral restraints is consistent with the current draft ASME B31 Mechanical Design Committee Appendix on Seismic Design (Ref. 103). Seismic experience data has indicated that relatively long spans have experienced lower spectral accelerations and are more susceptible to displacement-induced damage. Therefore, actual spans between lateral supports will often be limited to less than 4.5 times the B31.1 deadweight spans by Screen 6 (anchor motion of headers), Screen 9 (equipment nozzle loads), or Screen 12 (pipe support).

Lateral restraint may be provided either by an engineered lateral support, or by other means, such as:

1. Interferences

Lateral interferences will limit motion in piping routed along a wall or structural member. Although this restraint occurs in one direction only, it significantly restricts the response of the system to a reversing load.

2. Box Beam

A box beam, while not designed to provide horizontal restraint, will do so once the pipe moves through the gap and contacts the beam. When evaluating the effectiveness of a box beam's horizontal restraint potential, the gap on both sides of the pipe must be considered. Note that, should the pipe impact the vertical members of the beam, significant energy is dissipated and the frequency response of the system is modified.

3. U-Bolts

U-bolts provide significant horizontal restraint, even when the side load design capacity of the U-bolt is exceeded. Should the U-bolt yield under seismic stress, it will bend, resisting horizontal motion by tension. U-bolts should not be considered to provide longitudinal restraint along the pipe axis.

4. Saddles

There are generally two types of pipe saddle supports; a simple saddle on which the pipe merely rests, and that which includes a yoke (strap or U-bolt) to restrain the pipe in the saddle. A shallow simple saddle provides practically no horizontal restraint, and could permit the pipe to escape from its support during a seismic event. A deep saddle support will restrain the pipe in the lateral direction.

5. Floor and Wall Penetrations

Piping often passes through openings in floors, grating or walls. Since these openings are not designed as supports, gaps between the pipe and the structure exist. When made in floors or walls, the openings are usually secured by a sleeve; in gratings, a sleeve or a ring is used. These penetrations provide significant lateral restraint during dynamic seismic events and, like the box beam, prevent displacement, dissipate energy and modify system frequency.

6. Rod Hangers

The lateral support capacity of rod hangers is measurable as a function of the swing angle of the rod when subjected to a given lateral load. While this lateral support capacity is not provided by design, it can be important in practice. The length of the rod is significant because for shorter rods, the swing angle and resistance to horizontal displacement is greater. An effective lateral spring rate formula for short rod hangers is W/l, where W is the tributary weight on the rod and l is the length of the rod.

10.1.1.11 Anchor Motion (Screen 6)

Screen 6 - Piping must have sufficient flexibility to accommodate the seismic motions of structures, equipment and headers to which it is attached.

Commentary

One of the most common causes of piping failure in strong motion earthquakes is seismic anchor motion (SAM) resulting from:

1. large displacement of unanchored tanks or equipment

2. failure of the tank or equipment anchorage

3. large differential motions of structures to which the piping is attached

4. large motions of header piping induced into smaller branch piping

5. differential movements due to soil settlements

SAM caused by these sources imposes large strains in rigid sections of the piping system. Most of the common piping failures are in pipes with non-welded connections to tanks, pumps, and larger header pipes which are insufficiently restrained.

In order to screen out SAM as a potential failure mode for piping, the following conditions must be evaluated; otherwise the effect of anchor motion must be calculated.

1. Tanks and equipment to which the piping attaches must be properly anchored to prevent sliding, rocking or overturning. Equipment anchorage shall be evaluated using Chapter 6 and Section 9.1 of the DOE Seismic Evaluation Procedure.

2. Tanks and equipment to which the piping attaches, and the supports for the tanks and equipment should be relatively stiff to minimize SAM.

   Note: When vibration isolators are present, vibration isolators on equipment are a source of SAM, and must be evaluated as provided in Chapter 6 of the DOE Seismic Evaluation Procedure. If there are no seismic stops built into the isolators, the equipment will likely require the addition of seismic restraints to limit motion. If seismic stops are installed with the vibration isolators, the attached piping must be assessed for the maximum motion that can be realized before impacting the stops.

3. Piping rigidly attached to two different buildings, or substructures within a building, must be sufficiently flexible to accommodate the differential motion of the attachment points. Usually, structural displacements are relatively small, and the motion can be easily accommodated by pipe bending. Particular attention should be focused on piping that has its axial motion restrained at support points in two different structures

4. Header motion imposed on small branch lines must be assessed, or the header must be restrained near the branch.

The elastically calculated unintensified stress amplitude due to SAM (M/Z) may be limited to twice the material yield stress for screening purposes. When considering lateral movement of header pipes and restraint of branch pipes, it is necessary to define a lateral restraint, as discussed in Section 10.1.1.10, Lateral Span.

10.1.1.12 Mechanical Joints (Screen 7)

Screen 7 - Piping shall not contain mechanical joints which rely solely on friction.

Commentary

The seismic experience data contains a number of instances where mechanical joints which rely on friction have leaked. While it is not clear whether this leakage was due to seismic anchor motion effects (already covered by an earlier screen), these joints must be classified as outliers pending further studies. Joint vendors may be contacted or tests may be conducted to obtain allowable loads, and simple span formulas may be used to estimate applied loads to be compared to the allowables.

10.1.1.13 Flanged Joints (Screen 8)

Screen 8 - Flanged joints shall withstand the expected seismic moments without leakage.

Commentary

Flanged joints have leaked under severe seismic loads, and sometimes may leak under normal service loads. If the flanged joint is a B16.5 (Ref. 102) flange adequate preload, and a rated pressure above the operating pressure, the flange is acceptable. Other flanged joints with lesser capacities should not be located in high stress areas. One method of assessing moment capacity at flanges is to determine excess pressure capability (rating minus operating pressure) and convert that into an equivalent moment. The rated pressure of flanged joints shall be established.

If there are indications of leakage at the joint in past service, the flanged joint is an outlier.

Slip-on flanges are only acceptable if located in areas of the piping system with estimated unintensified seismic stress less than approximately 10,000 psi.

10.1.1.14 Equipment Nozzle Loads (Screen 9)

Screen 9 - Equipment shall not be subjected to large seismic loads from the piping systems.

Commentary

To be considered operable, active equipment and components (such as pumps and valves) have to meet the requirements of the DOE Seismic Evaluation Procedure (refer to Table 10.1.1-1), in addition to the following requirements:

Equipment and component nozzles, except for valves that are stronger than the pipe, should be protected, by appropriate restraints, from excessive seismic loads, particularly where the equipment nozzle or joint is of smaller size than the pipe. The piping layout shall be reviewed to evaluate that large seismic loads are not reacted at the equipment nozzle. One potential problem is a long axial run of pipe not restrained from axial movement except at the equipment nozzle. If there is a possibility of large seismic loads, the unintensified bending stress at the nozzle shall be elastically evaluated and compared to twice the material yield stress.

Piping reaction loads at the nozzles of rotating equipment may affect their function. The seismic reaction loads imparted by the piping on the nozzle of the active (rotating) equipment shall be estimated. These loads shall be small (unintensified bending stress less than 6000 psi), or within the estimated capability of the equipment.

10.1.1.15 Eccentric Weights (Screen 10)

Screen 10 - Eccentric weights in piping systems shall be evaluated.

Commentary

The adequacy of valves with eccentric operators shall be evaluated using the rules in Chapter 8 of the DOE Seismic Evaluation Procedure. Eccentric pipe segments, such as unsupported vents or drains, shall be evaluated using the peak spectral acceleration at 5% damping (or a better estimate of the spectral acceleration at the pipe frequency) (see Section 5.2) and an allowable unintensified elastically calculated stress of twice the material yield stress.

10.1.1.16 Flexible Joints (Screen 11)

Screen 11 - Flexible joints shall be properly restrained to keep relative end movements within vendor limits.

Commentary

For unsupported flexible joints such as expansion joints, bellows, or flexible joints, the relative displacements need to be limited to prevent tearing or buckling the joint. Where manufacturer's limits can be exceeded, the Review Team should ensure the joint has sufficient mobility to absorb the seismic deflections. When such joints are adequately supported on either side this is not usually an issue.

If the configuration is such that excessive seismic movements at the expansion joint could tear or buckle the joint, the expansion joint is an outlier. Calculation of seismic displacements and comparison to established allowable displacements are required to resolve the outlier.

The seismic evaluation team may refer to the rules of the Expansion Joints Manufacturers Association (EJMA).

10.1.1.17 Evaluation of Pipe Supports (Screen 12)

Screen 12 - Pipe supports shall be capable of withstanding seismic loads without failure.

Commentary

Support failure refers to non-ductile rupture or complete loss of restraining function of the pipe support.

The review team shall evaluate the seismic load and capacity of supports judged to be prone to failure. The basis for the support selection shall be documented.

Examples of supports to be evaluated are:

- supports with largest spans or close to heavy components
- supports reacting the load from long axial runs
- short rods adjacent to longer rods
- stiff support in the midst of significantly more flexible supports (hard-spot)

- supports with fewest or smallest anchor bolts
- gang supports reacting loads from several pipes
- supports not attached to structural steel or concrete (such a supports attached to other piping, cable tray or transite walls)

10.1.1.17.1 Seismic Demand

The calculation of horizontal and vertical seismic loading on pipe supports is based on the tributary weight of adjacent piping spans multiplied by one of the following factors:

1. For piping supported from grade, multiply by the peak of the 5% damped ground response spectrum. (see Section 5.2)

2. For piping supported above grade, multiply by the peak of the 5% damped floor (in-structure) response spectrum. (see Section 5.2)

10.1.1.17.2 Seismic Capacity

Where failure is credible, the review team shall evaluate the seismic capacity of support members along the seismic load path. The capacity of support members, welds and joints may be estimated using AISC (Ref. 81) rules, multiplying the AISC allowables by 1.7. Where manufacturer design limits are provided for standard pipe support elements (excluding anchor bolts in concrete), the seismic capacity may be taken as twice the design limit for members loaded in tension, bending or shear. For compression members, if the design limit is based on buckling, the seismic capacity shall be the same as the manufacturer design limit.

For cold-formed steel members, the stress allowables for seismic screening may be 1.7 times the AISI Specification for those members.

Anchorage shall be inspected, and capacity calculated and documented, using the rules of Chapter 6 of the DOE Seismic Evaluation Procedure.

The review team must take care to limit their calculations to credible failure modes which can hinder the function of the piping system. Limited yielding is, in most cases, not a credible failure mode.

An explicit calculation of weld capacities is not required if the welds are estimated to be the same size, and develop the same strength, as connecting members.

The fatigue capacity of threaded rod hangers with fixed-end connections to the wall or structural steel, may be evaluated using the fatigue evaluation screening charts for raceway supports in Section 9.2.1 of the DOE Seismic Evaluation Procedure.

10.1.1.18 Interaction with Other Structures (Screen 13)

Screen 13 - The piping being reviewed shall not be a source or target of interactions.

Commentary

A piping system subjected to seismic loads will displace or swing laterally, and may impact adjacent components.

10.1.1.18.1 Estimate of Displacement

Without detailed analysis, lateral displacements or swing deflections of piping spans can be estimated.

An approximate formula to estimate pipe displacements (Sd [in]) at spectral acceleration (S_a [in/sec^2]) for a pipe frequency f [1/sec], is:

$$S_d = 1.3\ S_a / (2\ \pi\ f)^2$$

where 1.3 is the mode participation factor for a simply supported beam. An approximate upper bound for a 0.3g Regulatory Guide 1.60 "Design Response Spectra for Seismic Design of Nuclear Power Plants" (Ref. 104) spectrum at low frequency (less than 0.25 Hz) is about 28" for 5% damping. Actual displacements of piping systems which meet the screens are rarely larger than 12".

10.1.1.18.2 Estimate of Impact Consequences

In all cases, the review team will have to carefully estimate the extent of pipe deflection and the component's capacity to absorb impact.

Generally, impact must be avoided if it affects the following components:

- active equipment (motors, fans, pumps, etc.)
- instrumentation
- tubing
- unstable or light weight structures
- electrical cabinets and panels
- sprinkler heads

Generally, impact may be of little consequence if it affects the following components:

- walls
- large frames or structures
- passive components (tank, check valve, etc.)
- pipes of approximately the same or larger diameter

In all cases, the review team must use judgment in estimating the extent of movement of the pipe under review and the capacity of the impacted equipment.

The review team shall visually inspect all structures and commodities located above the pipe and identify those hazards which are judged to be credible (may fall on the pipe) and significant (fall impact may cause pipe failure as defined in Table 10.1.1-1). The guidance in Chapter 7 of the DOE Seismic Evaluation Procedure for equipment interactions may be used for this evaluation.

10.1.2 UNDERGROUND PIPING

10.1.2.1 Scope

This section addresses the seismic evaluation of underground, single wall, pressure piping made of steel, ductile iron, or copper material. Pipe materials must be ductile at service temperatures. Ductile pipe behavior requires joints which are stronger than the pipe. Arc-welded or properly brazed joints are examples of ductile pipe design. Oxy-acetylene welded joints in steel pipes must be considered an outlier and evaluated in accordance with Section 10.1.2.6.

Single or double containment piping (comprised of a core pipe contained inside a buried jacket pipe, as is commonly the case for radioactive waste transfer lines) are covered in Chapter 7 of Reference 29 ("Seismic Design and Evaluation Guidelines for the Department of Energy High-Level Waste Tanks and Appurtenances," BNL 52361). This reference provides a rigorous methodology for evaluating underground piping. Additional guidance for evaluating underground piping is available in the "ASCE Guidelines for the Seismic Design of Oil and Gas Pipeline Systems" (Ref. 105) and ASCE 4 (Ref. 74).

Underground piping made of gray cast iron, non-ferrous alloys, welded aluminum, thermoplastics, fiberglass, reinforced concrete, and asbestos-cement may exhibit non-ductile behavior and must be considered an outlier. In addition, threaded joints, groove type mechanical joints, and flanged joints must be considered outliers as seismic induced displacements must be explicitly evaluated and compared to joint allowables. Mechanical joints which rely solely on friction are also considered outliers as they may have very low displacement capacity. Methods for dealing with outliers are described in Section 10.1.2.6.

10.1.2.2 Pipe Condition Assessment

The seismic evaluation of underground piping must include an assessment of the existing pipe condition with verification that there has not been significant degradation in the strength, ductility, wall thickness, and joint integrity. This assessment includes:

1. Confirmation of the compatibility of the pipe material, exterior coating, interior lining (where provided), with the conveyed fluid and the surrounding soil or backfill.

2. Examination of historical performance data and maintenance records for evidence of leakage or repairs.

3. A visual and volumetric examination of selected sections of the piping (which will have to be excavated at examination points) to confirm the soundness of materials and joints.

Should this assessment identify a problem with the existing pipe integrity, the piping should be considered an outlier. Piping designated as an outlier should be investigated over a larger extent of the pipe length than the selected sections to identify the entire extent of piping with the problem. Mitigation of piping integrity necessitates repair or replacement of the affected pipe length.

10.1.2.3 Applied Loads

Seismic loads acting on underground piping include wave passage directly inducing strains in the pipe, transient seismic anchor motion from differential movement of building or other structures to which the pipe is attached, and permanent seismic anchor motion from soil movements resulting from seismic induced liquefaction, lateral spreading, settlement, or landslides. Seismic loads are also induced by differential movement resulting from fault rupture intersecting underground pipe.

Concurrent non-seismic loads might include internal pressure, soil overburden and surface loads, thermal expansion, and natural soil settlements.

10.1.2.4 Evaluation of Piping for Wave Passage Induced Strains

Typically, underground piping constructed of ductile materials and ductile joints can safely withstand strains induced by wave passage effects during an earthquake. In addition, underground piping constructed of ductile materials and ductile joints can safely withstand transient differential movements of underground portions of buildings or other underground attachment points during seismic wave passage. In general, no explicit analysis is required in these cases. Analyses or detailed evaluation is required for the following cases:

- impedance mismatch between soils, such as soft soil to stiff rock
- bends in the piping at which there can be stress concentration effects
- piping which passes through the interface of a building to its supporting soil
- locations of excessive pipe corrosion

It should be noted that there is one reported case of seismic wave propagation induced pipe failure to a corrosion free modern continuous welded steel pipeline. This case study is described in Reference 106 in which it is believed that the case study is the only documented case of wave passage damage to modern welded steel underground piping. This case has very extreme parameters, as discussed in the following paragraph, which should be considered when evaluating underground piping for wave passage effects and designated underground piping as an outlier. It is unlikely that a similar combination of circumstances exist at a DOE facility.

The pipe, which is discussed in Reference 106, was damaged in the 1985 Michoacan Earthquake. The pipe was a 42 inch diameter, 5/16 inch wall thickness water pipe constructed in the early 1970's of API 120 X-42 grade steel (yield stress = 42 ksi). The pipe centerline was about 6.4 feet below the ground surface. The soil profile consists of 130 feet very soft clay underlain by two stiffer strata of 260 feet and 1300 feet thickness atop rock. The pipe failure was wrinkling and tearing of the pipe wall. Three factors contributed to the failure of this pipe (1) the ground motion was dominated by Rayleigh waves as the earthquake source was very distant from the pipe location; (2) the peak ground velocity was very high for the acceleration level as the observed PGV/PGA was about 170 in/sec/g instead of 48 in/sec/g given by Newmark for alluvium; and (3) the soil was extremely soft with a shear wave propagation velocity of only about 130 feet per second.

Other examples of ductile underground piping subjected only to seismic wave propagation have demonstrated very good pipe performance. It is judged that the one case of observed damage resulted from a very unusual combination of circumstances. If conditions approach those described for this case, the ductile pipe must be designated an outlier and appropriate analyses can be used to evaluate this piping.

10.1.2.5 Evaluation of Piping for Permanent Soil Movements

Underground piping at sites subjected to permanent soil movements due to settlement, lateral spreading, liquefaction, landslides, or fault displacement must be considered an outlier. In these conditions, the pipe must be evaluated in the manner described in Section 10.1.2.6.

10.1.2.6 Outlier Evaluation

Underground piping designated as an outlier must be explicitly evaluated for the ability of the pipe and joints to withstand seismically-induced soil movements, either transient wave passage effects or permanent ground movements. The preferred approach is to evaluate pipe deformations imposed during earthquake motion and associated effects and to compare to strain criteria developed from full scale pipe tests. In some cases, pipe stresses are evaluated and compared to empirically determined stress limits. Analytical techniques must account for non-linear pipe behavior as acceptable strains may be beyond the elastic limit. Analytical techniques must also account for the non-linear stiffness of the soil surrounding the underground piping.

A method for estimating pipe strains induced during earthquake wave passage is completely described in Chapter 7 of Reference 29. The approach involves estimating axial strain and curvature of the ground during seismic wave passage. These strains may be transferred to long straight runs of buried piping by friction or bearing. Strains (or stresses) at elbows, bends, and tees are then determined by pseudo-static beam on elastic foundation analysis subjected to the axial strain and curvature of the surrounding soil. In such an analysis, the piping system, including both straight and curved sections, are modeled by relatively simple beams supported by linear Winkler springs representing the confinement of the surrounding soil. Similar analysis may be used to determine pipe response due to transient differential movements of buildings or other structures to which the pipe is attached/anchored. By this approach, strains and stresses may be determined for straight pipe, elbow, bend, and tee configurations, and at joints. The resulting strains or stresses should be compared to allowable levels depending on the ductility and strength of the pipe material and of the deformation capacity of joints.

For underground pipe at sites subject to permanent differential soil movement, considerable effort must be expended to establish the amount of movement, the rate of movement, the direction of movement, and the area impacted by the movement. In such cases, the preferred solution is to mitigate the soil such that movements do not occur or to reroute the pipe to avoid the affected area. If this is not possible, underground pipe evaluation is typically performed by conducting analysis of non-linear representations of the pipe and surrounding soil subjected to conservative estimates of the permanent ground deformation caused by settlement, spreading, liquefaction, or landslide. The resulting pipe response is compared to empirically based pipe strain criteria. In some cases, it may be possible to evaluate the pipe using the pseudo linear beam on elastic foundation analysis described in Chapter 7 of Reference 29 and discussed above for wave passage effects. Guidance on the evaluation of underground piping subjected to fault displacement is provided in Reference 105. The allowable strain criteria in Chapter 7 of Reference 29 is more conservative than that in Reference 105.

10.2 MECHANICAL EQUIPMENT

10.2.1 HEPA FILTERS

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of HEPA Filters which are included in the Seismic Equipment List (SEL). The guidelines contained in this section are based on experience at Los Alamos National Laboratory as well as other DOE sites. Guidelines in this section cover those features of HEPA filters which experience has shown can be vulnerable to seismic loadings.

HEPA filters are generally used to prevent airborne radioactive material from being released to the environment. The environment may be a laboratory room, a facility, or external to a facility.

Filters attached to a glove box (see Figure 10.2.1-1) are used to limit the spread of radioactive material through out the ventilation system of a facility. By the "rule of the box" (see Section 2.1.3.4.1), these types of filters can be evaluated as part of the glove box. The evaluation of the equipment class of glove boxes is discussed in Section 10.2.2.

Filters which are used to scrub recirculating air in a facility or which scrub air that is released through the facility exhaust are generally found in filter plenums (see Figures 10.2.1-2 through 10.2.1-4). Filter plenums are generally similar to the equipment class of Air Handlers, which is discussed in Section 8.2.9, with the exceptions that there may not be a coil section and the fan may be external to the plenum structure. Therefore, the caveats given for Air Handlers in Section 8.2.9 can be used in the evaluation of HEPA filters. In addition, external fan units associated with filter plenums can be evaluated using the caveats given for the equipment class of Fans, as discussed in Section 8.2.10.

HEPA filters themselves are generally lightweight and firmly held in position to a frame by some type of restraining mechanism. Both the frame and the restraining mechanism need to be evaluated. The frame should be evaluated for overall stability and to determine if permanent deformations can take place that adversely affect the function of the filter bank. The restraining mechanisms should be reviewed to determine if the filters can come loose during an earthquake. Seismic evaluations should include not only the equipment the filters are installed in, but also the framing and restraining mechanisms within those pieces of equipment.

HEPA filters should also be reviewed for potential seismic interactions. One such interaction would be the effect of fire suppression water on the filter functionality. Should fire sprinklers activate during or following a seismic event and spray water on the HEPA filters, the HEPA will weaken and may fail to function as intended. In addition, should a seismic induced fire occur during or following an earthquake and the fire suppression fails to activate, heat from the fire could adversely affect the functionality of HEPA filters.

Figure 10.2.1-1 **HEPA filters are contained in stainless steel canisters bolted to the tops of these glove boxes.**

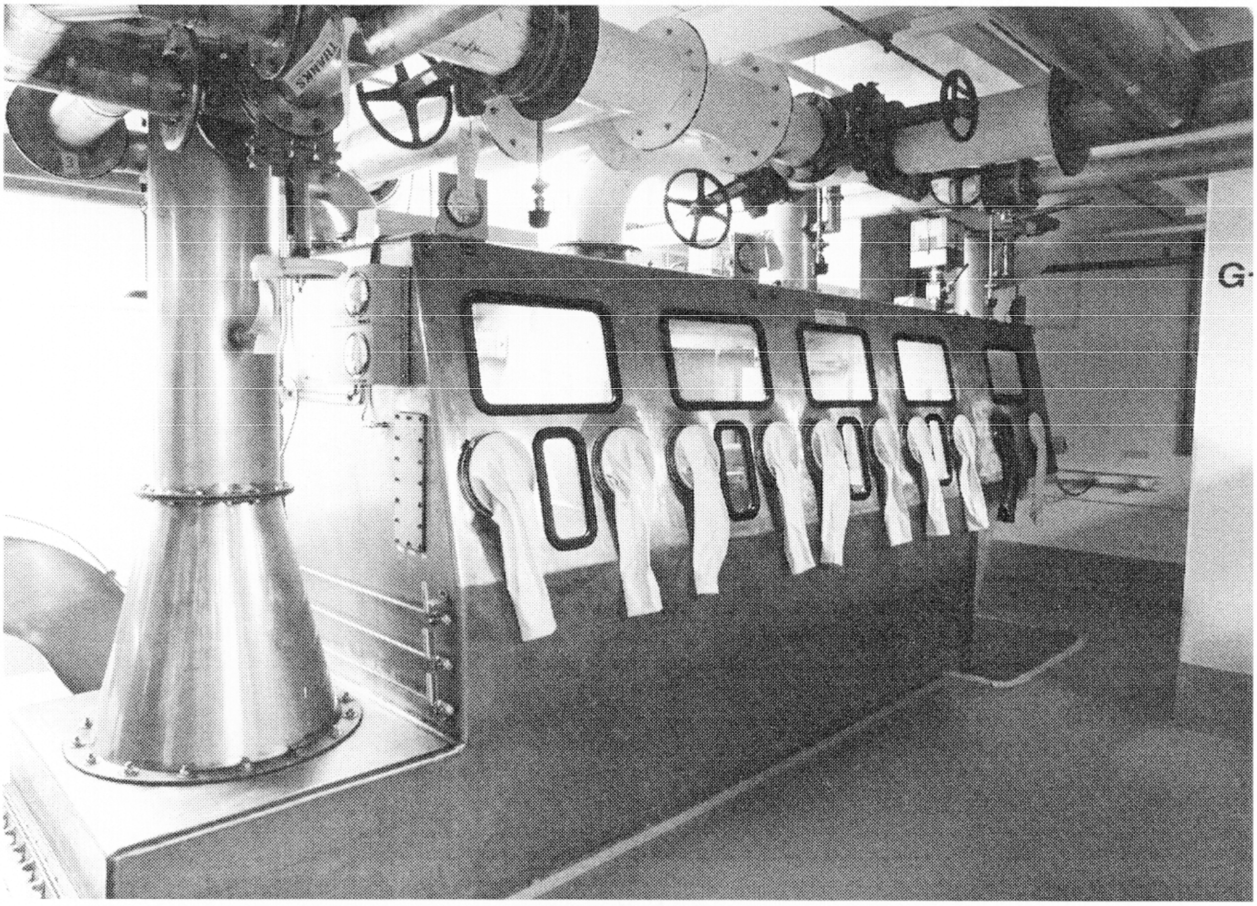


Figure 10.2.1-2 **This filter plenum containing a series of HEPA filters is similar to a glove box.**

Figure 10.2.1-3 **This filter plenum contains a series of HEPA filters and is constructed of structural steel tube frames with continuously welded steel plates for the walls, floor, and roof.**

Figure 10.2.1-4 HEPA filters (on the left side of the photograph) are securely held to the structural steel tube frame by bolted clamps (not shown). Also shown are dampers which are typically associated with filter plenums.

10.2.2 GLOVE BOXES

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of glove boxes which are included in the Seismic Equipment List (SEL). The guidelines contained in this section are based on analytical and walkdown experience at Los Alamos National Laboratory as well as other DOE sites. Guidelines in this section cover those features of glove boxes which experience has shown can be vulnerable to seismic loadings.

Glove boxes (see Figure 10.2.2-1) serve as primary confinement for radioactive or hazardous materials. As such, the pressure inside a glove box is less than the room pressure external to the glove box. Therefore, maintaining the pressure boundary is important when evaluating the seismic adequacy of glove boxes.

In evaluating glove boxes, the following five areas should be evaluated:

- seismic interaction effects, including flexibility of attached tubing and conduit and interaction with components or equipment located inside the glove box (heat sources, furnace, vacuum chamber, or flammable materials)
- load path
- supporting frame work
- leak tightness
- anchorage

As with other equipment, glove boxes are vulnerable to interaction effects. Windows, gloves and instrumentation tubing are all examples of fragile components associated with glove boxes that are prone to interaction effects. Interactions which should be considered include those that are both internal and external to the box. Externally, components such as power supplies and furnaces, which directly support glove box activities, should be restrained to prevent impact with windows (see Figure 10.2.2-2) and support frames. Internally, objects such as conveying systems and machining tools should be anchored to the box so that they cannot slide and tear gloves and break windows. Attached tubing and conduit need to have enough flexibility to accommodate the seismic motion of the glove box. Glove boxes which depend upon moment-resisting frame action for resistance of lateral seismic loads are more flexible than those using bracing and are therefore more susceptible to tubing and conduit failures. Additional guidance on evaluating the effects of seismic interaction is provided in Chapter 7.

The load path associated with the glove boxes needs to be evaluated. Load path refers to the manner in which inertial loads acting on the glove boxes and associated equipment are transferred through the glove box structure to the supporting framework, to the anchorage, and into the supporting structure. During seismic evaluations, the load path, including connections, should be carefully reviewed for adequate strength, stiffness, and ductility. Attachments, such as filtration devices and furnace wells, should be adequately anchored to the box. In addition, the box should be adequately attached to the supporting framework.

The supporting framework of glove boxes is one aspect of the evaluation in which structural calculations may be necessary to determine seismic adequacy. The framework should be reviewed for missing or altered (cutouts, notches or holes) members. Frames which rely on moment connections to provide lateral support and are constructed of unistrut or single angle legs have been found to be especially vulnerable. Braced frames are generally less vulnerable.

As previously noted, glove boxes serve as primary confinement for radioactive or hazardous materials. As such, leak tightness is an important feature of the glove box system. Interaction effects, load path, and supporting framework, in particular the relative displacements with connections boxes and attachments, could jeopardize the integrity of the pressure boundary associated with a glove box.

As with most equipment, anchorage should be evaluated using the procedure in Chapter 6. An area of concern which should be reviewed carefully is the gap between the bottom of the base plate and the floor. In many cases an individual glove box is part of a system or train of glove boxes in which one box is connected to another box. To maintain proper vertical alignment of the boxes, shims are typically used beneath the base plate (see Figure 10.2.2-3). These shims can introduce bending to the anchor bolts which can significantly reduce the capacity of the bolts. The reduction of bolt capacity due to bolt bending is briefly discussed in Chapter 6.

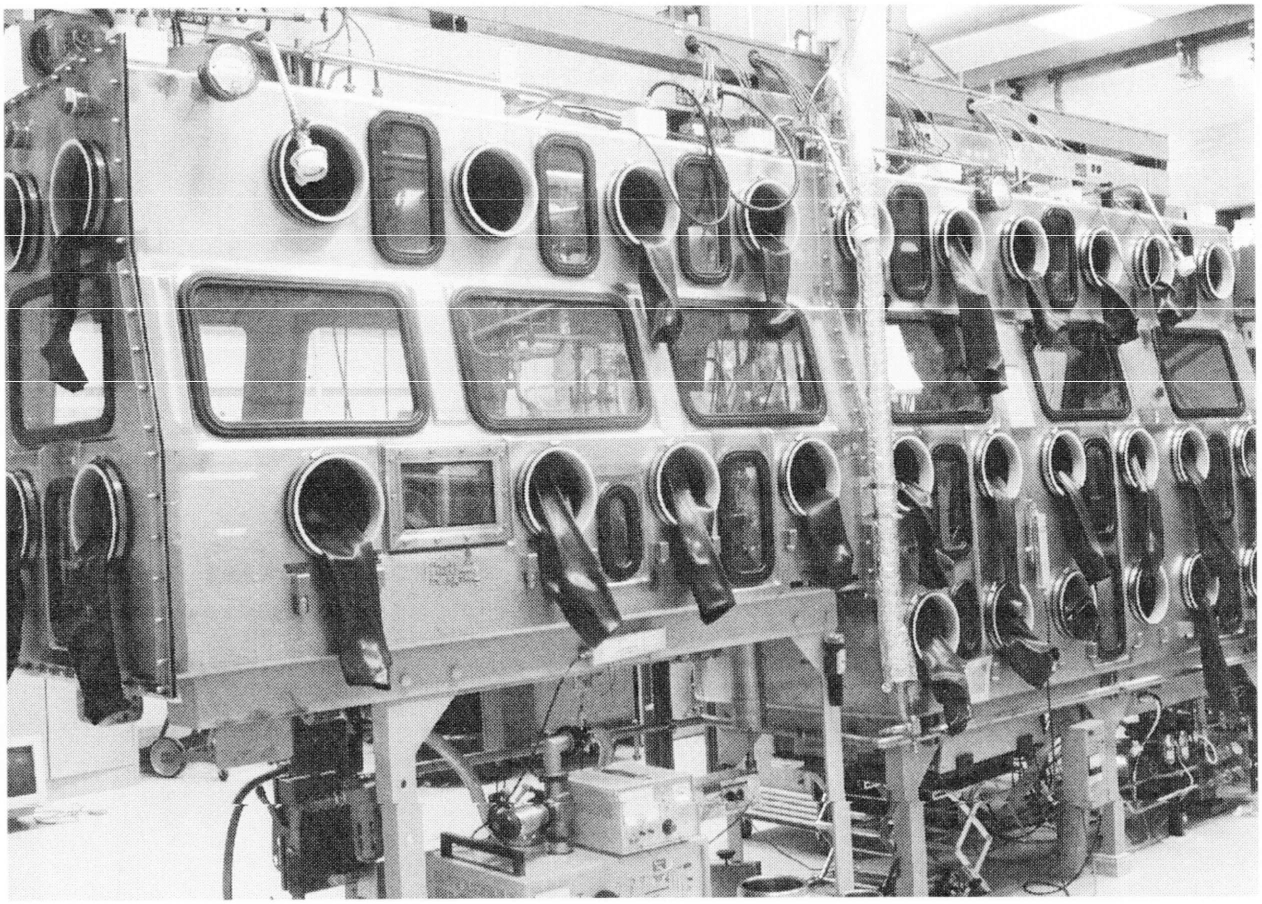


Figure 10.2.2-1 **Shown is a typical glove box. This particular glove box is supported by a moment resisting frame composed of single angle legs. Frames of this type have been found to be vulnerable to seismic loads.**

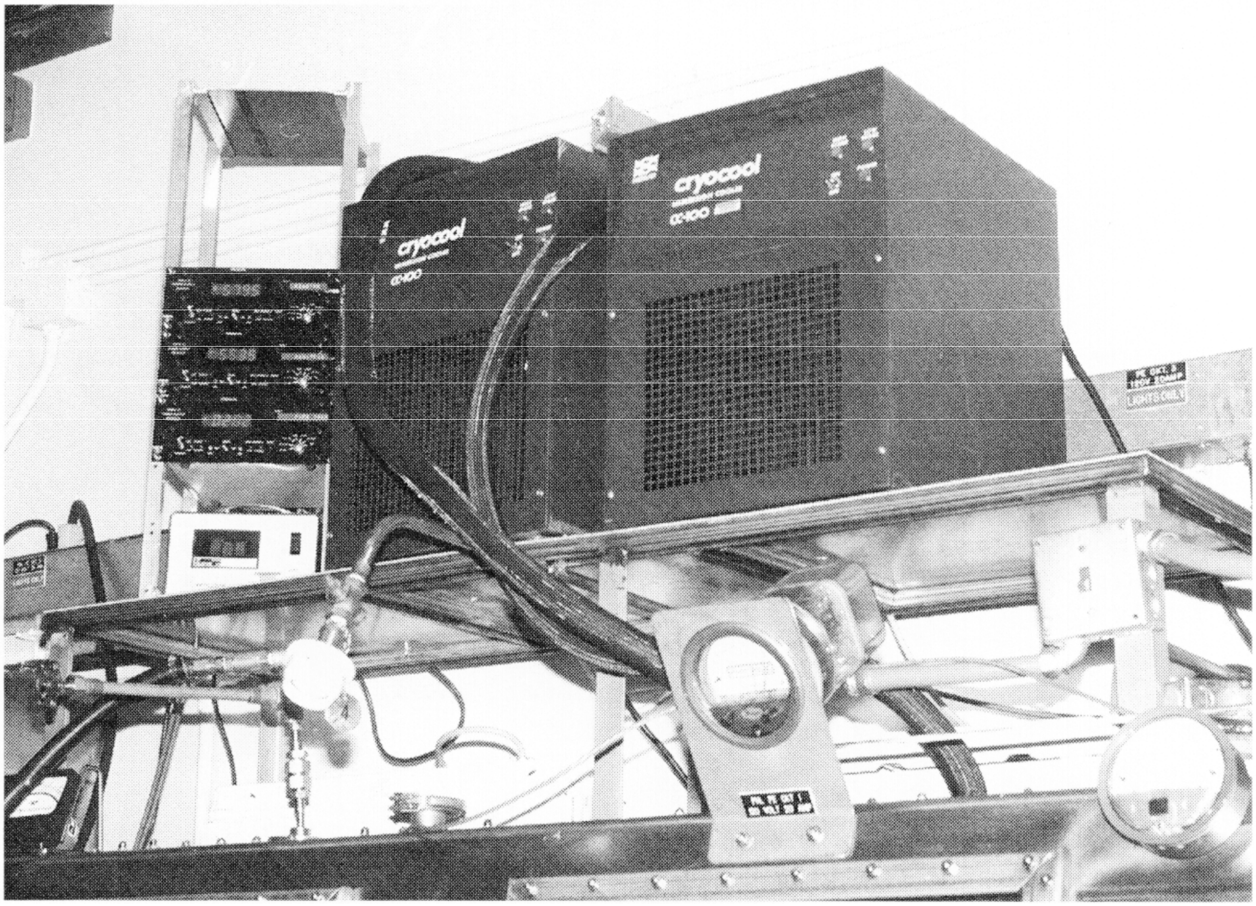


Figure 10.2.2-2 These refrigeration units support glove box activities. While the support stand is well supported on the top of the glove box, the units themselves are not anchored. During an earthquake, these units could slide off the support stand and impact a glove box window.

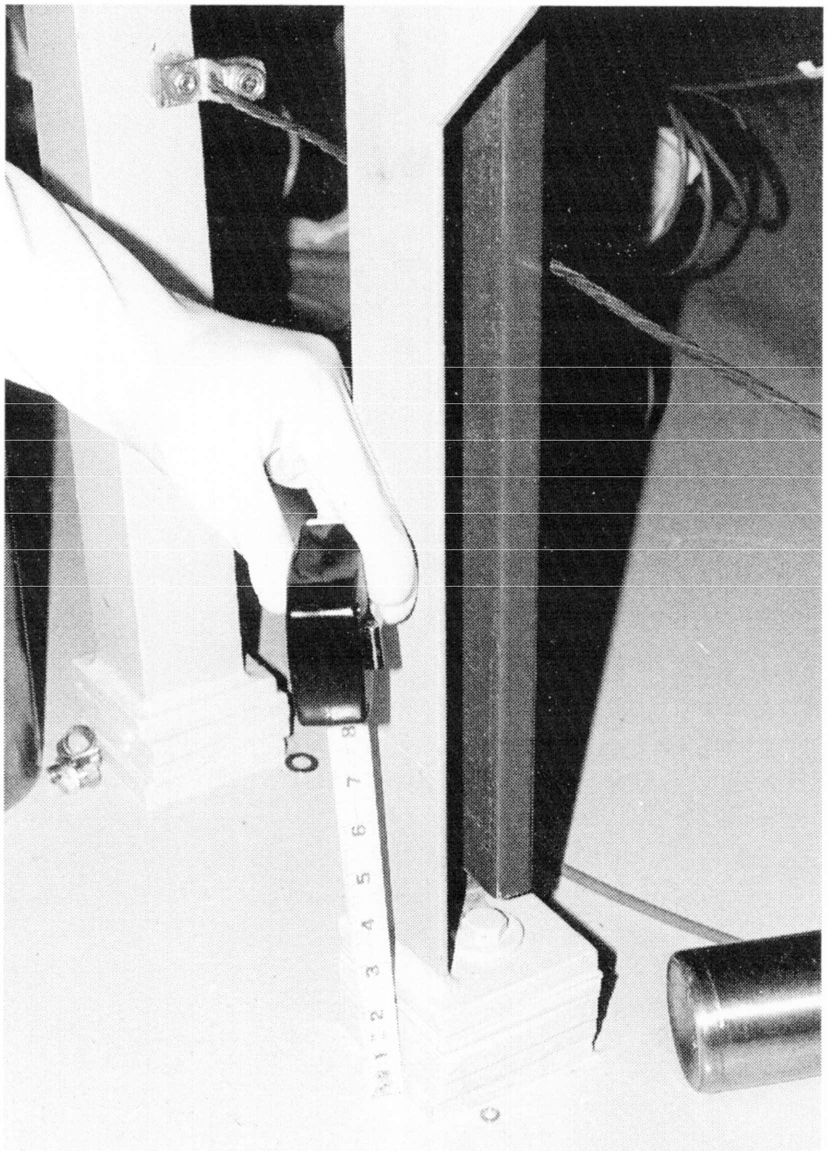


Figure 10.2.2-3 These legs have been shimmed to maintain proper vertical alignment of adjacent glove boxes. Excessive shim heights introduce bending to the anchor bolts which significantly decreases the bolt capacity.

10.2.3 MISCELLANEOUS MACHINERY

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of miscellaneous machinery which is included in the Seismic Equipment List (SEL). The guidelines contained in this section are based on Section 4.9 of "Practical Equipment Seismic Upgrade and Strengthening Guidelines" (Ref. 60). Guidelines in this section cover those features of miscellaneous machinery which experience has shown can be vulnerable to seismic loadings.

Miscellaneous machinery is typically contained in a machine shop or maintenance facility. The machinery types in the facility include: lathes (see Figure 10.2.3-1), band saws (see Figure 10.2.3-2), drill presses (see Figure 10.2.3-3), and work bench mounted machinery.

Industrial grade machinery, such as that shown in Figures 10.2.3-1 to 10.2.3-3, is typically very rugged and does not experience significant damage during an earthquake as long as it is well anchored. The rugged machinery typically has an adequate load path for earthquake-induced lateral loads. Unanchored or inadequately anchored components can be susceptible to sliding, overturning, or component misalignment as shown in Figure 10.2.3-4.

Three general methods of evaluating and providing anchorage for shop and mechanical machinery are outlined below. The screening evaluation for anchor bolts is provided in Chapter 6 with the miscellaneous machinery typically treated as rigid. For miscellaneous machinery, the seismic evaluation should emphasize its anchorage.

- Anchor bolts should be provided through existing holes in machinery base. Bolt sizes should be the same as the size of the furnished holes and excessive amounts of shims should not be used.

- For tall, narrow, and/or top-heavy machinery which may overturn in a strong earthquake, anchors should be provided at all four corners, as shown in Figure 10.2.3-5.

- For short, wide, and/or bottom-heavy machinery which may slide but not overturn, bumpers should be provided at all four corners. As shown in Figure 10.2.3-5, bumpers should contact the edges of the machinery if possible. A resilient pad, such as neoprene, may be glued to the face of the angle to reduce impact loads.

Many miscellaneous machinery components are box-like units that simply rest on a concrete floor. A minimum of four anchor bolts should be provided for each item and the spacing between the anchor bolts should not exceed 4 feet. For machinery provided with base plates or structural members with holes intended for anchors, expansion anchors should be provided in these holes. Otherwise, new clips or angle can be either welded or bolted to the machinery and expansion anchors provided for the floor. For tall machinery, anchorage to a wall with adequate capacity in addition to that provided at the base can greatly increase the seismic capacity of the anchorage system.

There are many installation conditions for machinery in a machine shop or maintenance facility. General categories of the conditions include machinery on skids or wheels. Approaches which may be used to evaluate and upgrade the machinery in the two categories are presented below.

<u>Machinery on Skids</u>

Skids supporting machinery should be structural steel (or equivalent structural material) and the skids should be anchored to the floor slab with the machinery anchored to the skid. Stiffener plates should be supplied for steel skids which support heavy machinery to provide adequate stiffness to resist seismically induced lateral loads. Some recommended anchorage approaches are presented in Figure 10.2.3-6.

Machinery on Wheels

A number of different types of machinery, including maintenance machinery and computer consoles, are supported on casters or wheels. Without proper lateral restraint, machinery on wheels can roll around and damage other property and/or injure personnel. Wheel locks and an appropriate temporary restraining system, such as chains, should be provided for machinery that must remain mobile for operational purposes. Tall machinery should be anchored to the wall or roof at the top to prevent overturning. For more permanent items, floor or wall anchors should be installed, as shown in Figure 10.2.3-7. When anchoring to an existing wall, the capacity of the wall and the details of the structural connection of the wall and roof should be evaluated. If the wall is an unreinforced masonry (URM) wall, the provisions of Section 10.5.1 should be used.

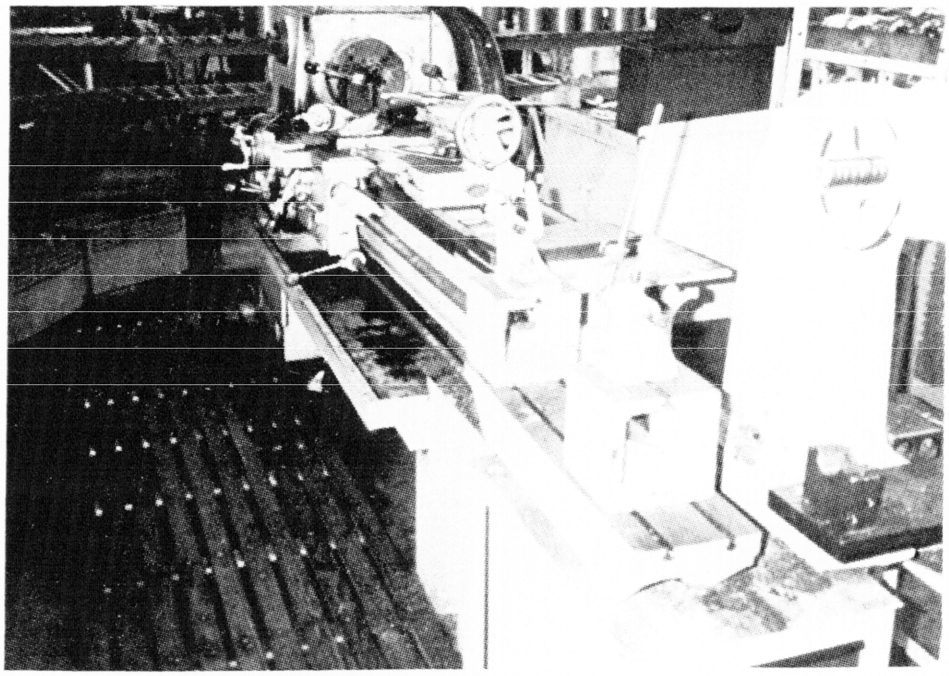


Figure 10.2.3-1 Unanchored Metal Lathe Susceptible to Sliding (Figure 4-69 of Reference 60)

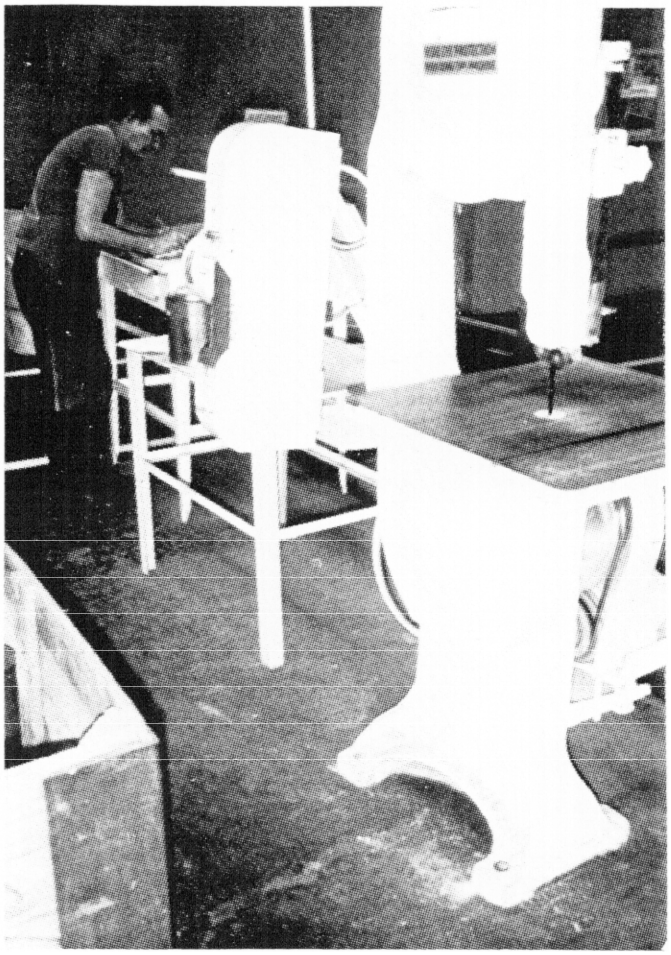


Figure 10.2.3-2 Anchored Band Saw (Figure 4-70 of Reference 60)

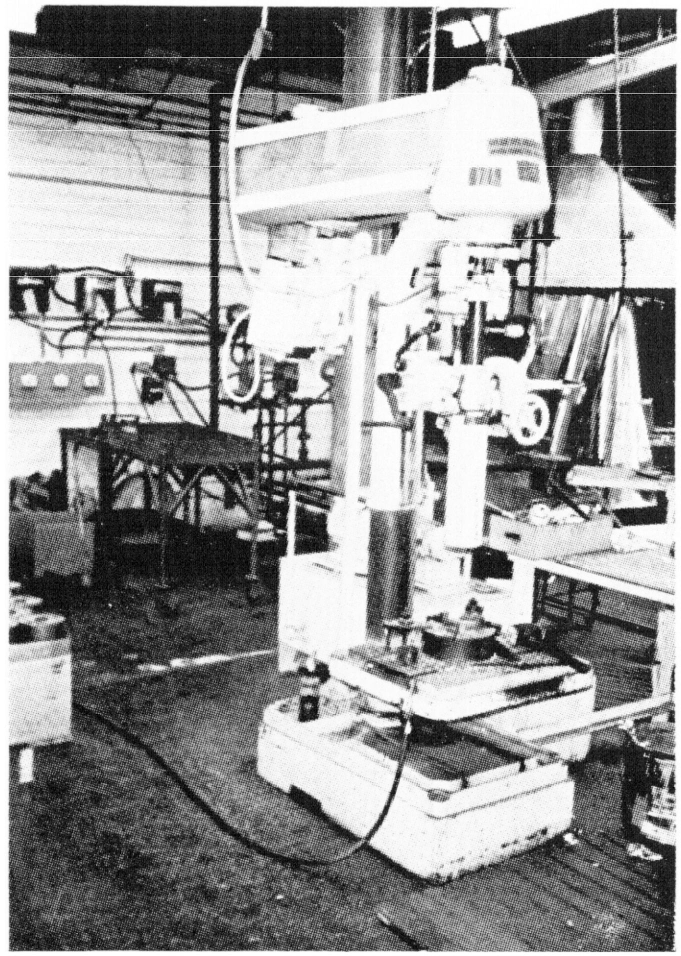


Figure 10.2.3-3 Unanchored Drill Press Susceptible to Overturning Damage (Figure 4-71 of Reference 60)

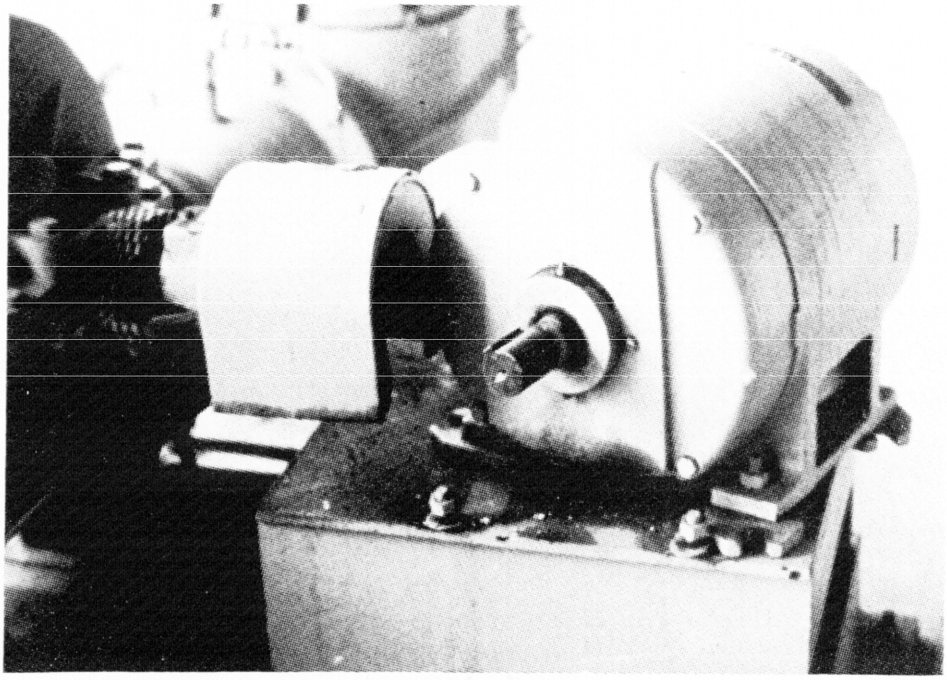


Figure 10.2.3-4 Misaligned Electrical Motor Resulting from Improper Anchorage (Figure 4-73 of Reference 60)

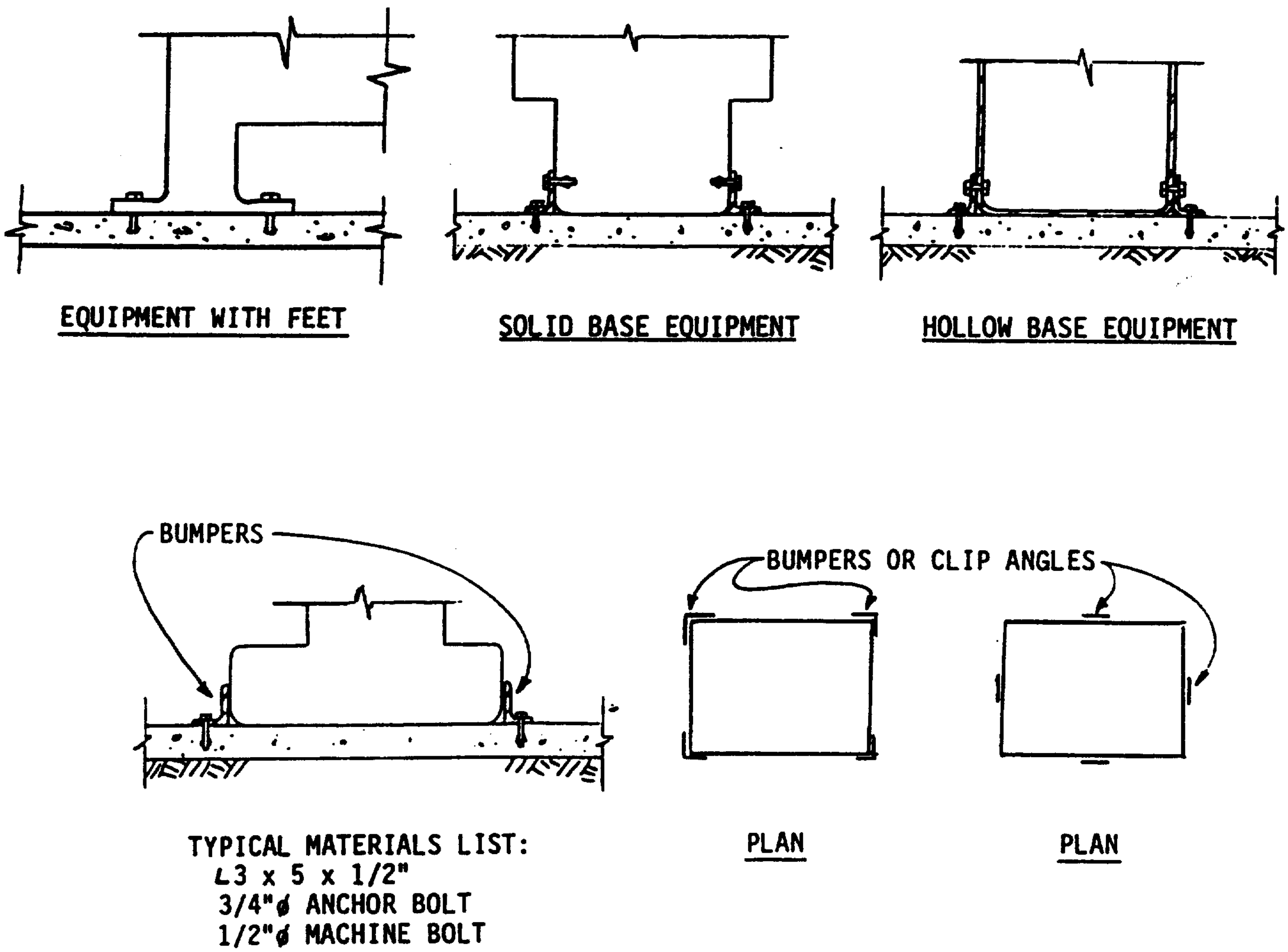


Figure 10.2.3-5 Approaches for Anchoring Machine Shop Equipment (Figure 4-74 of Reference 60)

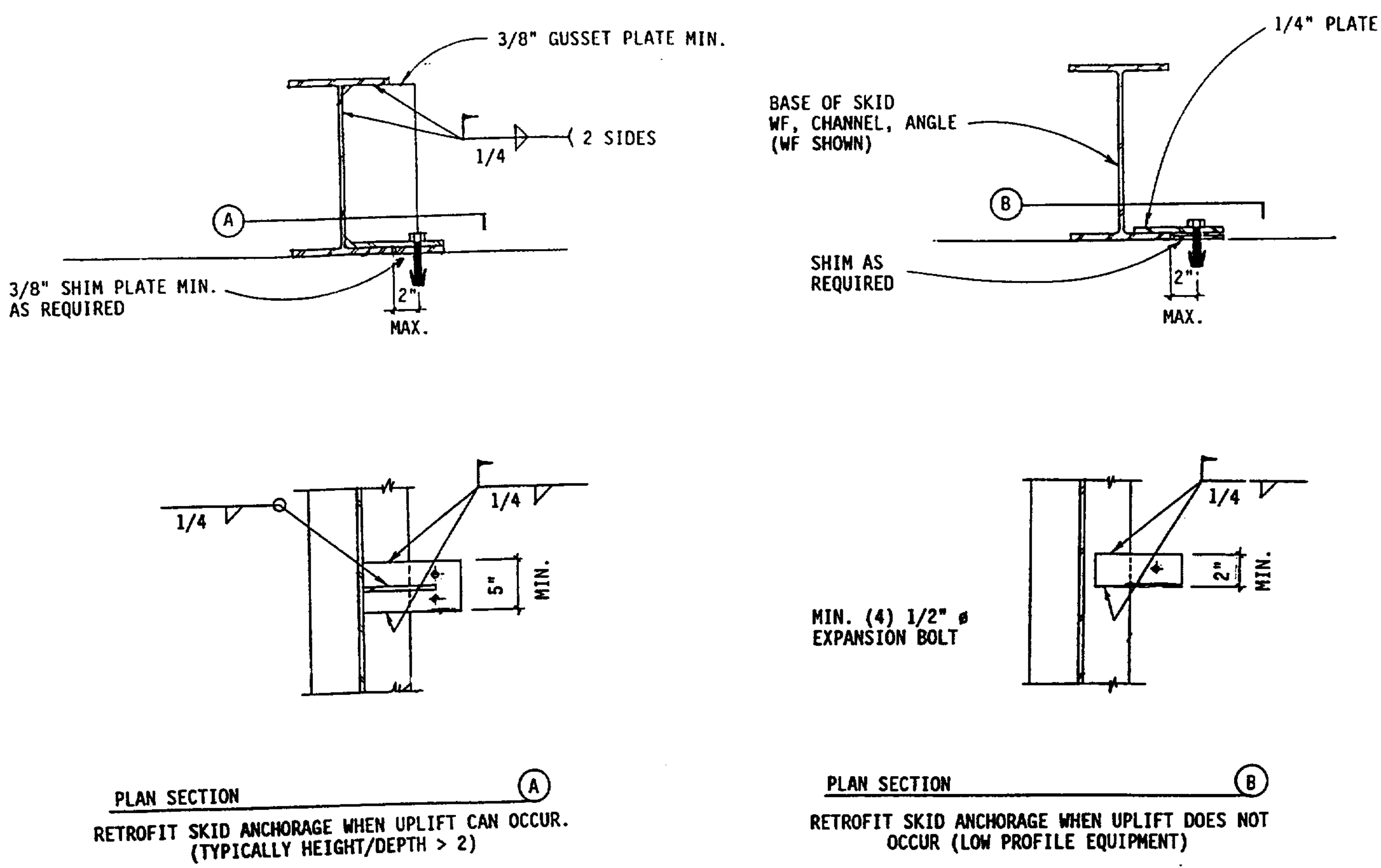


Figure 10.2.3-6 Approaches for Anchoring Equipment Skids (Figure 4-76 of Reference 60)

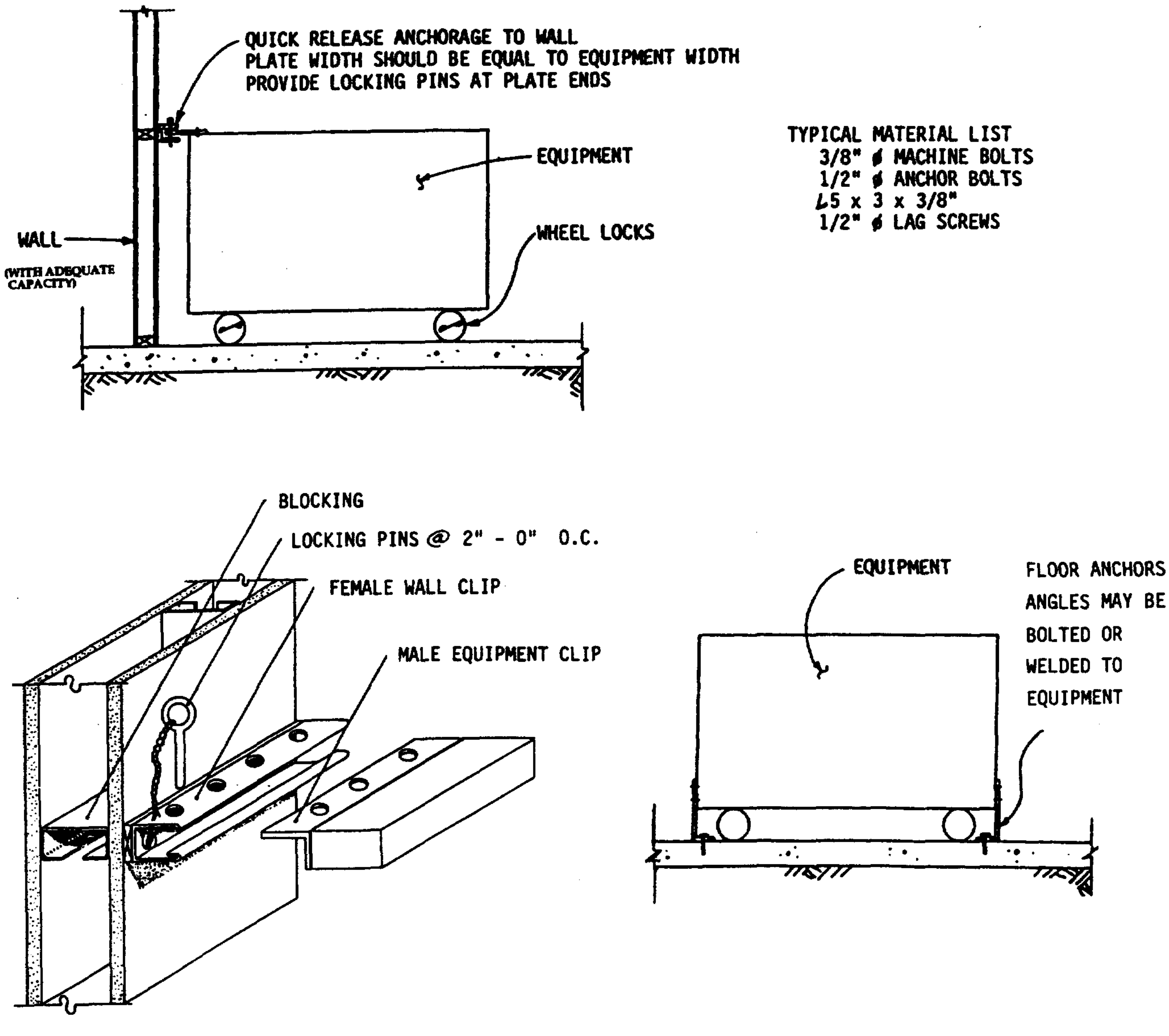


Figure 10.2.3-7 Approaches for Anchoring Equipment on Wheels (Figure 4-77 of Reference 60)

10.3 OTHER TANKS

10.3.1 UNDERGROUND TANKS

Guidelines for considering earthquake loading for the design and evaluation of underground storage tanks can be found in Reference 29 ("Seismic Design and Evaluation Guidelines for the Department of Energy High-Level Waste Tanks and Appurtenances", BNL 52361). This document was prepared for high-level waste tanks and specifically covers the primary tank, secondary liner, concrete vault, transfer piping, and the other components required to maintain the confinement function of a tank farm. The guidelines are developed primarily for double-shell tanks since it is expected that all new tanks will be double-shell structures. However, these guidelines are also generally applicable to single-shell tanks.

The design and evaluation guidelines in Reference 29 include a definition of the design basis earthquake ground motion, simplified methods for determination of soil-structure and liquid-structure interaction effects, analytical techniques for evaluating seismic demand, and criteria for assessing structural capacity. Table 10.3.1-1 provides a road map to the various subjects addressed in Reference 29. The abstract states that these guidelines reflect the knowledge acquired in the last two decades in the areas of defining the ground motion and calculating hydrodynamic loads and dynamic soil pressures, and other loads for underground tank structures, piping, and equipment. Interpretation and implementation of the guidelines are illustrated through examples.

Table 10.3.1-1 Use of "Seismic Design & Evaluation Guidelines for the Department of Energy High-Level Waste Tanks & Appurtenances" (Ref. 29)

Subject Matter	Chapter and/or Appendix from Reference 29
Seismic Design and Evaluation Criteria	Chapter 3
Evaluation of Tank Response	
Hydrodynamic Effects	Chapter 4
Liquid Viscosity Effects	Appendix B
Soil-Structure Interaction	Chapter 6 and Appendix H
Effect of Top Constraint	Appendix C
Seismic Response Example	Appendix G
Evaluation of Tank Capacity	
Seismic Capacity	Chapter 5
Inelastic Energy Absorption	Appendix A
Buckling of Tanks	Appendix F
Effects of Sloshing Striking the Roof	Appendix D
Dimension Tolerance and Fabrication Details	Appendix E
Associated Structures and Equipment	
Underground Piping (Section 10.1.2)	Chapter 7 and Appendix I
Equipment Qualification	Chapter 8

As described in Chapter 3 of Reference 29 (see Table 10.3.1-1), the seismic guidelines for underground storage tanks are based on the same target performance goals upon which general seismic design and evaluation criteria for Department of Energy structures, systems, and components as given in DOE-STD-1020 (Ref. 6) are based. Deterministic, pseudo-linear seismic evaluation procedures are provided that are based on the DOE target performance goals. The document recognizes that there may be situations where explicit non-linear dynamic analysis of structures or soil columns may be necessary. It also recognizes cases, such as liquefaction analysis, where there may not be existing capacity standards consistent with the deterministic procedures. For these situations, a more general approach for complying with the target performance goals is discussed in which alternative design or evaluation techniques may be employed.

In addition to general seismic design and evaluation criteria, many subjects specifically addressing issues pertinent to underground storage tanks are covered by Reference 29 as illustrated by Table 10.3.1-1. In general, these subjects include evaluation of hydrodynamic effects in tanks, seismic capacity of tanks, evaluation of soil-vault interaction, and underground piping and conduits. Each of these areas is briefly described in the following paragraphs.

A critical element in the analysis of the seismic response of the tank-liquid system is the evaluation of the hydrodynamic pressures exerted against the tank wall and base. Once these pressures have been established, the corresponding forces and stresses in the tank may be determined with relative ease. Methods of evaluating hydrodynamic pressures for horizontal, rocking, and vertical components of earthquake ground motion are presented. In addition, sloshing motion of the free liquid surface is considered. These items are addressed in Chapter 4 and Appendix G of Reference 29 (see Table 10.3.1-1). Of special interest for waste storage tanks are the effects of inhomogeneous liquids within the tank or the influence of liquid viscosity on hydrodynamic effects which is addressed in Appendix B of Reference 29.

Assessment of the seismic capacity of tanks in Reference 29 considers observed failure modes for tanks in past earthquakes. Flat bottom vertical liquid storage tanks have sometimes failed with loss of contents during strong earthquake shaking. For tanks with radius to wall thickness ratios greater than about 600 or tanks with minimal or no anchorage, failure has often been associated with rupture of the tank wall near its connection to the base, due either to excessive tank wall buckling or bolt stretching and excessive base plate uplift. Both failure modes are primarily due to the dynamic overturning moment at the tank base from fluid pressure on the tank wall. Other common failure mode have been breaking of piping connected to a tank as a result of relative movement and severe distortion due to a soil failure (soil liquefaction, slope instability, or excessive differential settlement). Other failure modes, which are of much lesser importance either because of their general lack of occurrence or less severe consequences, but which deserve some attention, are: tank sliding, excessive hoop tensile stresses due to hydrodynamic pressures on the tank wall, damage to the roof due to insufficient freeboard for fluid sloshing, and damage to internal attachments from lateral and torsional fluid movements. Tank capacity evaluation is addressed in Chapter 5 and Appendices A, F, D, and E of Reference 29 as shown in Table 10.3.1-1.

Important considerations for soil-vault interaction are evaluation of the seismic input motion to the support points of the tank and the seismically induced pressures on the walls of the vault. Evaluation of soil-vault (soil-structure) interaction must consider the vertical spatial variation of the free field ground motion and that the motion of the vault may differ from the free field motion. Guidelines for necessary soil properties and evaluation of soil structure interaction effects applied to underground tanks are presented in Chapter 6 and Appendix H of Reference 29.

Most underground waste process piping systems are encased (or double containment) piping systems. The inner pipe serves to transport the wastes and maintain the pressure boundary and the outer pipe provides secondary containment and is in direct contact with the surrounding soil. The design of underground piping systems and conduits must demonstrate the ability of the piping system to withstand strains and stresses caused by potential seismic movement of the surrounding soil in conjunction with stresses induced by other concurrent loads. Guidelines are provided to consider different aspects of seismically induced ground movements including: (1) abrupt relative displacements of the ground at faults; (2) ground failure of relatively large areas caused by liquefaction, landslides, gross surface movements, or collapse of voids at depth; (3) transient deformation of the ground during the earthquake due to wave passage effects; (4) inertial response of the inner piping system in response to induced movements of the outer piping; and (5) transient movements of anchor points or buildings connected to buried facilities. As shown in Table 10.3.1-1, underground piping is addressed in Chapter 7 and Appendix I of Reference 29.

10.3.2 CANISTERS AND GAS CYLINDERS

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of canisters and gas cylinders which are included in the Seismic Equipment List (SEL). Guidelines in this section cover those features of canisters and gas cylinders which experience has shown can be vulnerable to seismic loadings.

Unanchored compressed gas cylinders will tip over at very low levels of ground shaking. If the reducing valve should snap off, the canister may become a high speed missile. In addition, escaping gas may represent a potential fire, explosion, or toxic gas hazard to nearby personnel.

Compressed gas cylinders often have a single safety chain located about mid-height (Figure 10.3.2-1). A single chain is not sufficient to prevent tipping during an earthquake. Examples of properly anchored cylinders are presented in Figures 10.3.2-2 and 10.3.2-3. In these figures, the gas cylinders have upper and lower safety chains, or restraints.

In the event of an earthquake, poorly restrained canisters and gas cylinders may fall and roll, spilling their contents, causing damage to other equipment, and/or injuring personnel. Methods of restraining them, including providing positive anchorage to a wall, storing them in well braced and anchored racks, or storing them horizontally on the floor, are shown in Figure 10.3.2-4. The supports for the canisters should be attached to walls that have adequate capacity to resist the seismic demand from the canisters. Adequate capacity typically results from two levels of support or a structural storage system that restrains moments.

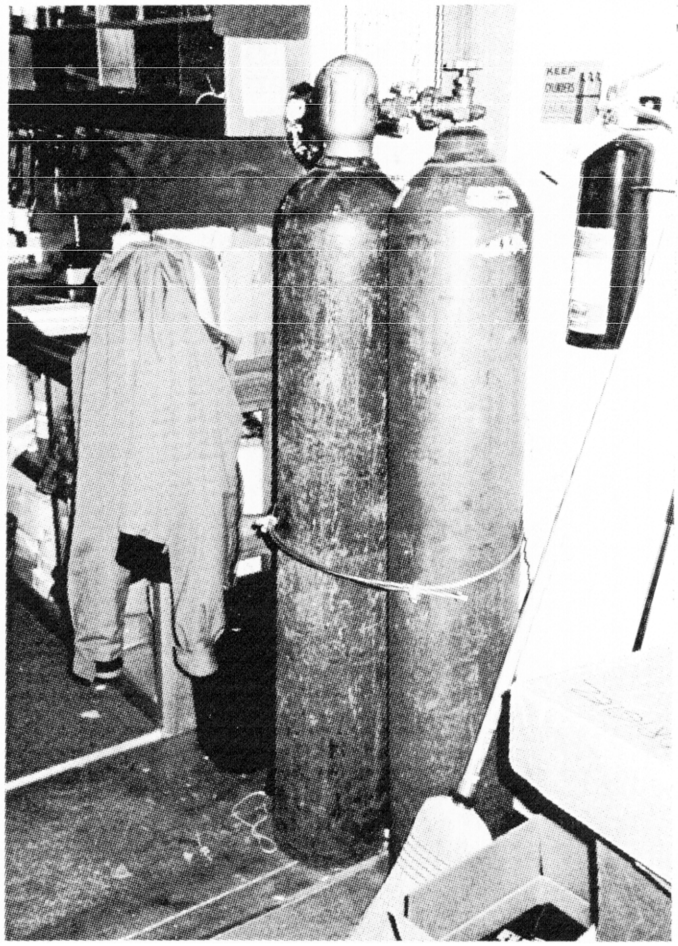


Figure 10.3.2-1 Compressed Gas Cylinder that is Inadequately Anchored with a Safety Chain Located at Midheight (Figure 4-55 of Reference 60)

Figure 10.3.2-2 Adequately Anchored Compressed Gas Cylinder (Figure 4-57 of Reference 60)

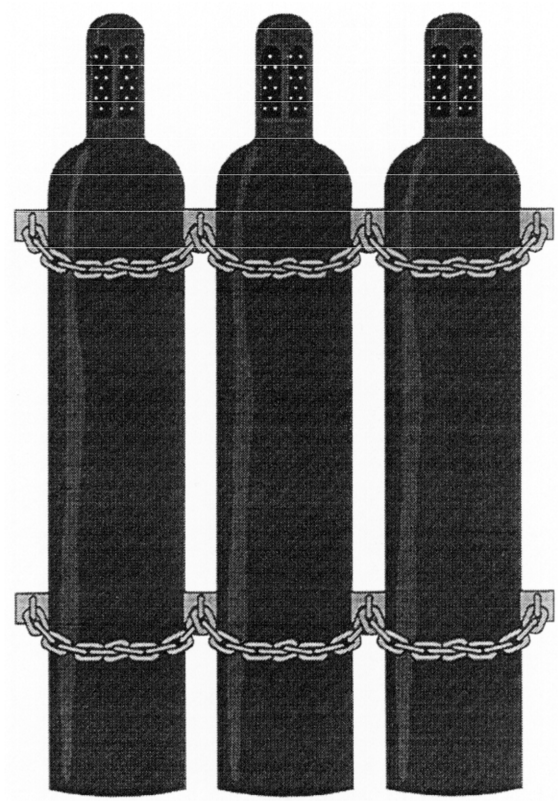


Figure 10.3.2-3 Upper and lower restraints are required for gas bottles.

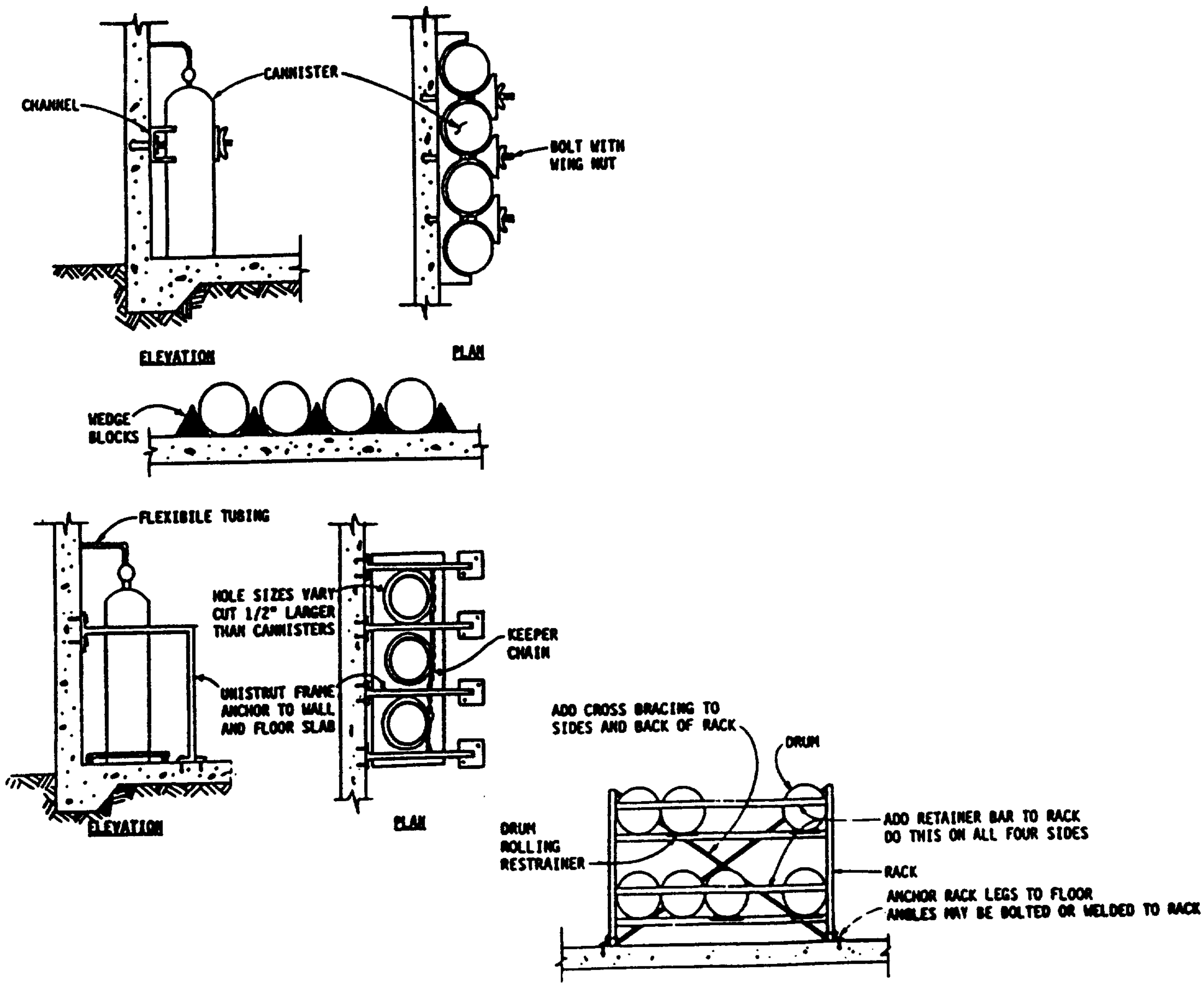


Figure 10.3.2-4 Approaches for Anchoring Canisters (Figure 4-56 of Reference 60)

10.4 DUCT SYSTEMS

10.4.1 HVAC DUCTS

This section is the "Procedure for the Seismic Evaluation of Steel HVAC Duct" (Ref. 28) which was developed by the Westinghouse Savannah River Company and is based on information in Reference 107. It is limited to applications involving existing duct systems. For new design, the engineer is referred to other methods documented in References 108, 109, and 110. Additional information is contained in References 111 and 112.

10.4.1.1 Scope

This procedure provides seismic evaluation rules for existing rectangular or round steel HVAC duct. The objective of this evaluation procedure is to ensure a high confidence of acceptable seismic performance for the following:

- duct structural integrity
  - material condition
  - joint, seam, and stiffener design
  - vertical and horizontal support bracing
  - heavy components and appurtenances
  - stiff branches
- duct pressure boundary integrity (if applicable)
  - joint, seam, and stiffener design
  - duct panel stress
  - duct support bearing (point contact)
  - flexible bellows
- duct support integrity
  - material condition
  - seismic capacity vs. demand
  - support anchorage
  - support details (load path)
- seismic interactions

The duct system seismic evaluation includes facility walkdown reviews and limited analytical reviews of bounding sample configurations. The relationship and typical sequence of these reviews is shown in a logic diagram in Figure 10.4.1-1.

Fans (including louvers) and air handlers (including dampers) are covered in Sections 8.2.10 and 8.2.9, respectively.

Duct mounted dampers that are not part of the fan or air handler assemblies and floor mounted filter housings and plenums must be evaluated separately and are not covered by this procedure.

10.4.1.1.1 Industry Standards

"HVAC Duct Construction Standard, Metal and Flexible", SMACNA (Ref. 113)

"Rectangular Industrial Duct Construction Standards", SMACNA (Ref. 114)

"Round Industrial Duct Construction Standards", SMACNA (Ref. 115)

10.4.1.1.2 Duct Loads

(a) Duct weight

(b) Coating and insulation weight

(c) Positive (outward) or negative (inward) uniform pressure. Typically expressed in inches of water gage (wg), as a differential pressure relative to atmosphere (1 atmosphere = 0 wg and 1 wg = 0.0361 psig).

(d) Weight of particulate accumulation in the duct.

(e) Weight of workmen or implements resting from time to time on the duct.

(f) Forces due to wind, for outdoor duct.

(g) Forces due to seismic events.

(h) Vibration from system operations.

Loads (a) through (f) are addressed in the SMACNA design standards.

Seismic loads (g) are evaluated by this procedure which is based on design standards, testing and seismic experience as documented in Reference 107.

Vibration loads (h) are typically evaluated and resolved after system start-up.

10.4.1.1.3 Seismic Review Team

The seismic review team shall consist of a minimum of 2 engineers certified in the use of the DOE Seismic Evaluation Procedure (see Section 3.2) and knowledgeable in the design requirements of the SMACNA standards. They shall document their review on Screening Evaluation and Work Sheets (SEWS) as described in this procedure. Each evaluation attribute shown on the SEWS form is described in Section 10.4.1.2 through 10.4.1.6 of this procedure.

10.4.1.1.4 Duct System Boundary

The duct system boundary establishes the scope of the configuration to be evaluated. These boundaries are determined based on consideration of system requirements and operational needs during or following a seismic event. For example, the HVAC system performance requirements following an earthquake may be to support environmental confinement of hazardous materials. In this case, pressure boundary integrity is important. The HVAC evaluation boundaries may terminate at system isolation points such as dampers. Furthermore, the evaluation scope might be limited to portions of the system that support filtration (e.g. HEPA which is also discussed in Section 10.2.1) and effluent exhaust.

In some cases, the performance objective of the HVAC system may be to convey air for the comfort and safety of building personnel. In this situation, duct structural integrity is the primary objective (instead of a high degree of pressure retention).

A Screening Evaluation Work Sheet (SEWS 10.4.1) may encompass a single run of duct, a duct system (several runs of duct with the same operating parameters) or a group of duct systems. The SEWS should describe, by sketch or system identification, the scope of ducts covered.

10.4.1.1.5 Evaluation Objectives (Pressure Boundary/Structural Integrity)

Where only structural integrity is required, some leakage in or out of the duct is allowed, provided the duct retains its spatial configuration and does not fall. This procedure addresses the seismic structural integrity of the duct and its support system together with a review for potential seismic interactions.

Where pressure boundary integrity is required, the duct wall can not be breached and the duct joints and seams must remain pressure tight. An example is that of a HVAC duct that is used for conveyance of hazardous effluent gas to a HEPA filter. In general, confinement HVAC systems are configured so that the operating pressure for the hazardous gas is maintained at a negative pressure relative to the environment of the duct exterior. The safety requirements for such a configuration have very limited tolerance for duct leakage so as to preserve the duct system effectiveness and efficiency. Consequently, this duct would probably be classified as a safety related item (PC3 or PC4). This procedure augments the duct structural integrity evaluation requirements with additional criteria to provide a high degree of confidence that pressure boundary integrity will be maintained during a seismic event.

10.4.1.1.6 Functionality Requirement

HVAC duct systems may be required to function during a seismic event. In this case, spurious changes of equipment condition (such as accidental closing or opening of dampers, or loss of controls) are not permitted to occur.

HVAC duct systems may be allowed to malfunction during the period of seismic vibration, provided it can be reset (remotely or by local manual controls) to function after the seismic event.

10.4.1.1.7 Bounding Sample Evaluation

A group of duct systems may be evaluated based on a worst-case bounding sample review. For each attribute, the Seismic Review Team must select the worst-case configurations. For example, the review for stiffener spacing may be based on panels having the largest width, thinnest gage, greatest distance between stiffeners with the smallest section properties. The basis for the selected bounding sample(s) should be documented on the SEWS form.

10.4.1.2 Evaluation for Structural Integrity

10.4.1.2.1 Duct Free of Damage, Defects, Degradation

The HVAC duct system network should be visually inspected for damage, defects, and degradation. The inspection should also identify suspect repairs, missing parts, broken joints, poor workmanship and significant corrosion, particularly at duct joints.

10.4.1.2.2 Duct Material and Stiffeners Comply with SMACNA

A visual inspection of the ducts should confirm that the duct material, stiffeners and joints are in accordance with SMACNA (Ref. 113, 114, and 115).

In particular, the following attributes must be verified:

a. Materials should be rolled steel (below 650°F operating temperature), galvanized steel (below 400°F), or stainless steel.

b. Stiffeners should comply with SMACNA: steel shapes (Ref. 113 Page 1-23 and Ref. 114 Page 7-56 to 58), angle or bar reinforcement for round ducts (Ref. 115 Page 4f-2). Fastening of the stiffener to the duct should be by tack weld, spot weld, bolt, screw or rivet, 12" max. spacing (Ref. 113 Page 1-48).

10.4.1.2.3 Duct Joints and Seams Comply with SMACNA

Joints and seams should conform to SMACNA standard configurations, and be positively attached, excluding friction or riveted joints. Acceptable transverse joint configurations are: Ref. 113 Page 1-35; Ref. 114 Page 8-7; Ref. 115 Pages 5-4 and 5-11 excluding sleeved (Figure 3), riveted (Figure 4) and draw band (Figure 5) joints. Acceptable longitudinal seam configurations are groove weld and fillet weld (Ref. 114 Pages 8-1 through 8-6; Ref. 113 Page 3-5), and lock type (Ref. 113 Page 1-40) excluding riveted seams.

10.4.1.2.4 Duct Meets Support Span Criteria

10.4.1.2.4.1 SMACNA Rules

SMACNA provides rules for the spacing of duct supports (Ref. 113 Page 4-3, Ref. 114 Page 9-7, and Ref. 115 Page 7-3), based on a maximum allowable bending stress in the duct wall of 8 ksi for rectangular duct and 10 ksi for circular duct.

For seismic loads, the same spacing criteria must be met, however an increase of the allowable bending stress by 33% is allowed provided the duct joints are type T-17 to T-24 (Ref. 113 Page 1-35).

10.4.1.2.4.2 Computing the allowable support span length for rectangular duct:

The SMACNA approximation for rectangular duct section properties is based on four 2" corners (Ref. 114 Page 9-7) and a bending stress ($\sigma = w L^2 / 10$ which is based on the average of simply supported and built-in moment). For duct with uniformly distributed load, the allowable span between consecutive vertical supports can be expressed as:

$$L = [80 F_b / (H + W) \rho K_R]^{1/2}$$

where:

F_b	=	allowable bending stress (psi) [typically 8000 psi for rectangular duct]
H,W	=	height, width of duct (in) (see Figure 10.4.1-2)
ρ	=	equivalent density of duct material (lb/in^3). (Note - Include insulation and reinforcement mass contribution).
K_R	=	parameter for rectangular duct in Section 10.4.1.6.1 (1/in^2)

10.4.1.2.4.3 Computing the allowable support span length for circular duct:

The SMACNA approximation for circular duct is based on a bending stress $\sigma = w L^2 / 10$. For circular duct with uniformly distributed load, the corresponding allowable span between consecutive vertical supports can be expressed as:

$$L = (5 F_b D / 2 \rho K_c)^{1/2}$$

where:

F_b	=	allowable bending stress (psi) [typically 10,000 psi for circular duct]
D	=	duct diameter (in)
ρ	=	equivalent density of duct material (lb/in^3). (Note -Include insulation and reinforcement mass contribution).
K_c	=	parameter for circular duct in Section 10.4.1.6.2 (dimensionless).

10.4.1.2.4.4 Effect of concentrated weights

Heavy in-line components, such as unsupported in-line dampers subject to seismic accelerations, exert an additional bending moment on the duct. The allowable support span must be reduced accordingly, to limit the bending stress to within the allowable F_b.

Beam equations may be used to superimpose the distributed weight and the concentrated weight stress (see Section 10.4.1.6.3 for additional guidance).

10.4.1.2.5 Duct Guided Against Sliding Off Supports

Seismic experience indicates that HVAC duct can fail if it slides off its supports. The duct must be secured, by tie-downs or stops, if it can slide and fall off its supports.

10.4.1.2.6 Heavy In-Line Components Properly Restrained

Components mounted in-line on the duct work include fans, coolers, dryers, dampers, motor operators to dampers, and blowers.

In-line equipment must be positively attached to ductwork. Duct connections to heavy in-line components must be evaluated for structural capacity.

Support spans are to be reduced for heavy in-line components as discussed in Section 10.4.1.2.4.4.

In-line floor mounted equipment on vibration-isolation pads requires a separate evaluation based on failures recorded in the experience database. Guidance in performing this review is given in Chapter 6.

10.4.1.2.7 Appurtenances Properly Attached

Appurtenances to ducts include dampers, louvers, diffusers, and screens. Appurtenances must be positively attached to duct work (such as screwed or riveted) as opposed to slipped into place.

Duct connections to heavy cantilevered appurtenances must be evaluated for structural capacity.

10.4.1.2.8 No Stiff Branch With Flexible Headers

Branch ducts must have sufficient flexibility to accommodate potential sway movement of a flexibly hung duct header.

In particular, the review should identify lateral duct branches rigidly supported off long runs of duct with no axial restraints. The axial movement of the header could damage the branch duct. Similarly, a duct on sway type supports (such as rod hung trapeze or rod hangers) could swing and rupture a rigidly supported branch duct.

10.4.1.3 Evaluation for Pressure Boundary

Duct which has to maintain a pressure boundary must meet all of the screens for structural integrity (Section 10.4.1.2) and the following supplementary requirements.

10.4.1.3.1 Duct Joints and Seams Are of Rugged Type.

In addition to the criteria for structural integrity (Section 10.4.1.2), transverse joint configurations T-1 to T-16 (Ref. 113 Page 1-35) are outliers for pressure boundary review. Similarly, all longitudinal seams that are not groove or fillet welded (Ref. 114 Pages 8-1 through 8-6; Ref. 113 Page 3-5) are considered to be outliers for pressure boundary review.

10.4.1.3.2 Stiffeners and Joints Welded or Bolted to Duct

Duct stiffeners and joint reinforcements shall be attached to the duct by intermittent welds or by bolts with a maximum spacing of 12". For rectangular duct, the maximum distance of a weld or bolt from the duct edge is 2", (Ref. 113 Page 1-48).

Intermittent welds are typically staggered on alternate sides of the stiffeners and shall be 1" to 3" long (Ref. 114 Page 7-55).

10.4.1.3.3 Duct Gage, Stiffeners Sized to Resist Seismic Load

The Seismic Review Team shall verify the adequacy of the duct wall thickness (gage), stiffener size, and stiffener spacing in accordance with SMACNA (Ref. 113, 114, and 115), with the following provisions:

(a) The seismic accelerations generate uniform pressures acting on the duct in both + (internal pressure) and - (external pressure) directions. Due to the small deflections in duct wall, the scaled 2% damped accelerations must be used to evaluate stresses in duct walls.

(b) The stiffener deflection limits in SMACNA may be exceeded under seismic loads, provided the stiffener and the duct wall remain elastic. The SMACNA equations (Ref. 114 and 115) or the theory of plates and shells (Ref. 116) may be used for the stress analysis.

10.4.1.3.4 No Potential for Puncture of Duct Wall

Duct should not be supported on sharp edges or have point contacts with support members. Duct should be sufficiently restrained in the vertical and lateral directions, in accordance with the support span criteria for structural integrity, to avoid sliding or uplift impact.

10.4.1.3.5 Flexible Bellows Can Accommodate Motions

Where flexible bellows are provided, potential seismic displacements must be compared to bellows capacity. Alternatively, the bellows must be guided to preclude significant seismic differential movements.

10.4.1.4 Support Review

10.4.1.4.1 No Broken, Defective, or Degraded Hardware

Duct supports shall be visually inspected for adequate fabrication and maintenance. Signs of poor construction quality or subsequent degradation include: distortion, dislodged or shifted support members, missing brackets, nuts or bolts, unusual or temporary repairs, cracks in concrete, etc.

10.4.1.4.2 Support Member Capacity Exceeds Demand

The Seismic Review Team shall evaluate the sample support configuration(s) likely to have the largest demand/capacity ratio.

10.4.1.4.2.1 Seismic Demand

Ductile Supports: HVAC duct supports suspended from overhead or sidewalls (i.e. not supported from the floor) and which can be classified as ductile, as defined in the DOE Seismic Evaluation Procedure, must be evaluated for vertical capacity. The demand shall be based on 5 times the dead load in the downward direction (Ref. 107 page 39). A high vertical capacity provides considerable margin for horizontal earthquake loading.

Non-Ductile Supports: HVAC duct supports not classified as ductile, must be evaluated for vertical and horizontal (lateral or axial) loads. The scaled 7% damped peak spectral acceleration should be used to calculate applied loads, unless the spectral acceleration (see Section 5.2) at the duct span resonant frequency is determined.

$$F_a = W A_S$$

where:

W = tributary weight (lbs)
A_S = spectral acceleration (g)

Base-mounted supports represent a special type of non-ductile support. They are different than suspended supports in that base-mounted supports can become unstable when subjected to excessive lateral deflections or inelastic behavior since they don't have the pendulum restoring force attributes of suspended supports. Consequently, base-mounted support evaluations should include P-delta effects if there is the potential for base hardware slip. P-delta effects represent the second order increase in base overturning moment due to additional eccentricity of supported dead load during seismic deflections of the support. It is illustrated in Figure 10.4.1-3. Base plate flexibility (rotation) shall be postulated as applicable according to the following:

- shell expansion anchor slip of 1/8"
- channel nut slip of 1/16"
- clip angle bending

Additional discussion of base mounted support evaluations for P-delta effects is found in References 47 and 50.

10.4.1.4.2.2 Seismic Capacity

The support capacity shall be based on AISC (Ref. 81) including provisions to increase seismic allowable stresses by 1/3 (Ref. 81 Part 5 Section 1.5.6) and evaluation of potential for buckling.

HVAC duct supports consisting of rod hangers with fixed end connections shall be evaluated for fatigue (Ref. 47).

10.4.1.4.3 Anchorage Adequacy

For the bounding sample support configuration(s), the Seismic Review Team shall evaluate the support anchorage in accordance with Chapter 6 of the DOE Seismic Evaluation Procedure.

Anchor bolt installation (tightness) checks shall be performed for floor mounted supports as per Chapter 6 as well.

10.4.1.4.4 Support Details

Supports shall not include design details which have been a source of failure in past earthquakes such as beam clamps with no restraining strap, smooth channel nuts (without teeth or ridges) and cast-iron inserts.

10.4.1.5 Seismic Interaction Review

An evaluation shall be performed of potential seismic interaction hazards due to spatial proximity and differential motion between structures. Other seismic interaction evaluation considerations are identified in Chapter 7.

Free from Input by Nearby Equipment - Duct systems adjacent to other equipment should be evaluated for the consequences of interaction with moving items.

No Collapse of Overhead Equipment, Distribution Systems, or Masonry Walls - Duct Systems attached to or in the vicinity of unanchored components or unreinforced block walls should be evaluated for potential interaction.

Able to Accommodate Differential Displacements - Duct systems that span between different structures shall be evaluated to ensure adequate flexibility to accommodate relative movement of the structures.

10.4.1.6 Span Factors and Concentrated Weights

10.4.1.6.1 Span Factor for Rectangular Duct

Horizontal run of duct:

$$K_R = \left\{ S_h^2 R^4 W^2 \Big/ \left[(W^2/2) - W + 1) \right]^2 + S_v^2 H^2 \Big/ \left[\left(H^2/2 \right) - H + 1 \right]^2 \right\}^{1/2} + H \Big/ \left[(H^2/2) - H + 1 \right]$$

where:

S_h = horizontal spectral acceleration (see Section 5.2), lateral to duct (g)
S_v = vertical spectral acceleration (g) (see Section 5.2)
R = ratio of horizontal to vertical support spacing
W = width of duct (in)
H = height of duct (in)
K_R = span factor ($1/in^2$)

Vertical run of duct:

$$K_R = \left\{ S_{hW}^2 R^4 W^2 \Big/ \left[(W^2/2) - W + 1) \right]^2 + S_{hH}^2 H^2 \Big/ \left[(H^2/2) - H + 1 \right]^2 \right\}^{1/2}$$

where:

S_{hW} = horizontal spectral acceleration (see Section 5.2), parallel to side W (g)
S_{hH} = horizontal spectral acceleration (see Section 5.2), parallel to side H (g)
R = ratio of lateral support spacing in S_{hW} direction 2 to lateral support spacing in S_{hH} direction 1
W = width of duct (in)
H = height of duct (in)
L = maximum allowable support span in S_{hH} direction 1 (in)
K_R = span factor ($1/in^2$)

10.4.1.6.2 Span Factor for Circular Duct

Horizontal run of duct:

$$K_c = 1 + (S_v^2 + R^4 S_h^2)^{1/2}$$

where:

R = same as for horizontal rectangular duct
S_v = same as for horizontal rectangular duct
S_h = same as for horizontal rectangular duct
K_C = span factor (dimensionless)

Vertical run of duct:

$$K_c = (S_{h1}^2 + R^4 S_{h2}^2)^{1/2}$$

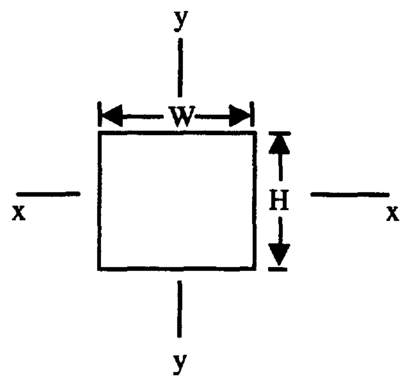
where:

S_{h1}	=	horizontal spectral acceleration (see Section 5.2) in direction 1
S_{h2}	=	horizontal spectral acceleration (see Section 5.2) in direction 2
R	=	ratio of lateral support spacing in direction 2 to lateral support spacing in direction 1
L	=	maximum allowable support span in direction 1 (in)
K_C	=	span factor (dimensionless)

10.4.1.6.3 Stress Equation

Seismic and weight bending stress in a duct due to its distributed weight and the weight of a heavy in-line (duct-mounted) component located mid-span is given below. For a horizontal rectangular duct, the stress is computed to be:

$$f_b = \left(wL^2/10 + PL/6\right)\left\{1 + \left[\left(a_H W/2I_{yy}\right)^2 + \left(a_v H/2I_{xx}\right)^2\right]^{1/2}\right\}$$



where:

f_b	=	total bending stress (psi)
w	=	distributed wt of duct (lbs/in)
L	=	length of duct span containing concentrated weight (in)
P	=	concentrated weight (lb)
a_H, a_v	=	horizontal and vertical accelerations (g)
W,H	=	width and height of duct (in)
I_{xx}, I_{yy}	=	moment of inertia of duct cross section (in^4). xx axis is parallel to width W; yy axis is parallel to height H (see figure above)
R	=	ratio of horizontal to vertical support spacing = 1

For a horizontal circular duct, the stress is computed using the above equation with W = H = D where D = outer diameter of duct (in).

10.4.1.6.4 Moments of Inertia for Rectangular Duct

Based on the SMACNA rectangular duct approximation of 4 corner angle sections (Ref. 114 Page 9-7), the moment of inertia is:

$$I_{xx} = 4\,t\,(H^2 - 2H + 2) \qquad (in^4)$$

$$I_{yy} = 4\,t\,(W^2 - 2W + 2) \qquad (in^4)$$

where:

t	=	duct thickness (in)
H	=	width of duct (in)
W	=	height of duct (in)

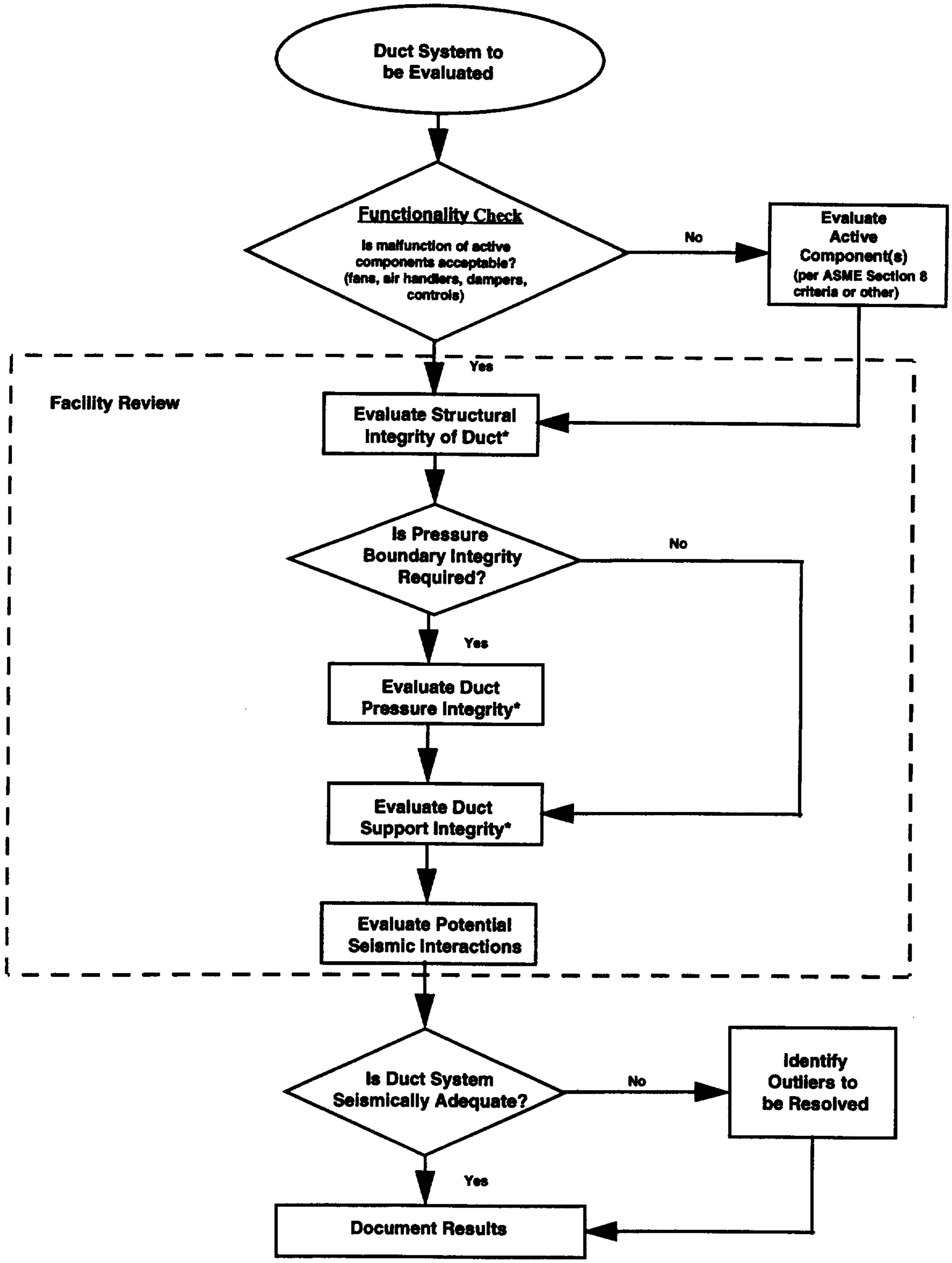
Note that the above equations include the 2" x 2" corners; hence, the H and W units must always be inches. If either W or H exceeds 72 in., the corresponding value used for calculating I_{xx} and I_{yy} shall be 72 in. Moment of inertia and section modulus calculations shall be based on dimensions $\leq$ 72 in. (Ref. 114 Page 9-7).

10.4.1.6.5 Moments of Inertia for Round Duct

$$I_{xx} = I_{yy} = 0.0491\,(D^4 - d^4) \qquad (in^4)$$

where:

D	=	outer diameter of duct (in)
d	=	inner diameter of duct (in)



* Requires limited analytical review in addition to field walkdown.

Figure 10.4.1-1 Logic Diagram for Duct System Seismic Evaluation

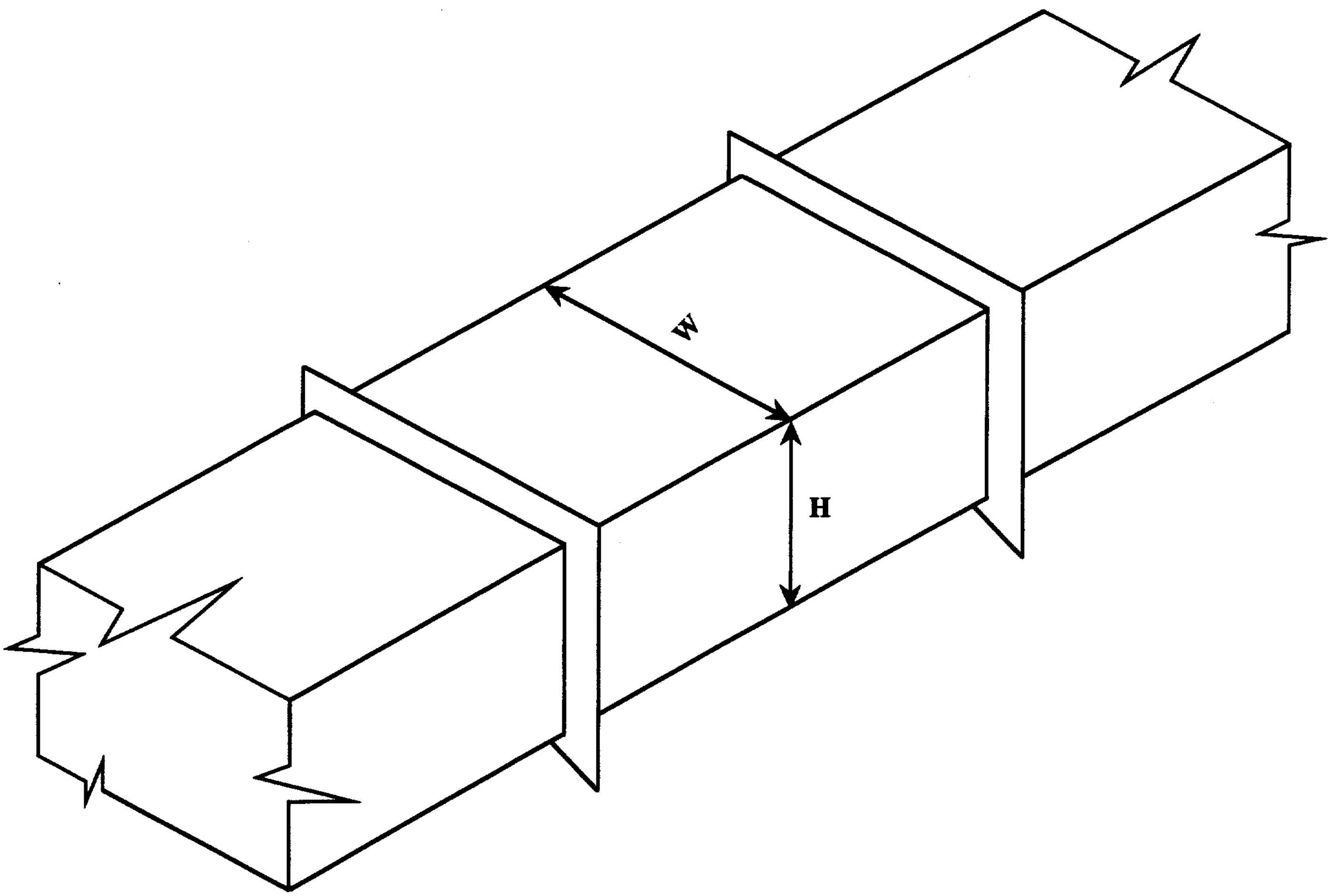
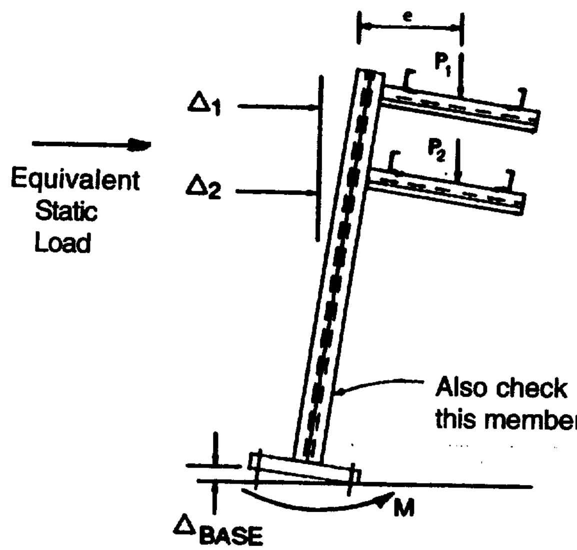
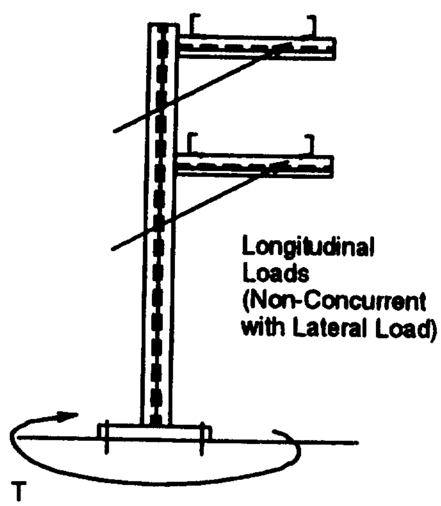


Figure 10.4.1-2 Typical Rectangular HVAC Duct Section



P-Δ EFFECTS FROM
BASE ROTATION ONLY



NEGLECT TORSION FOR
LONGITUDINAL LOAD

Figure 10.4.1-3 Floor Mounted Supports

10.5 ARCHITECTURAL FEATURES AND COMPONENTS

10.5.1 UNREINFORCED MASONRY (URM) WALLS

This section provides guidance in evaluating unreinforced non-bearing masonry (URM) walls for seismic adequacy. It should be noted that the approaches presented herein address only the out-of-plane behavior of <u>non-bearing</u> unreinforced masonry walls with respect to seismic loads. It is important to have a list of masonry walls selected before the Seismic Review Team (SRT) begins its seismic evaluation. The Seismic Capability Engineers (SCEs) that make up the SRT are not necessarily the ones expected to assemble the list of selected masonry walls for evaluation. That is a separate task to be performed by others (see Chapter 4).

The selected masonry wall is first examined by non-destructive evaluation (NDE) methods to determine if it is hollow or grouted solid. If the wall is found to be hollow in every cell (or only hollow in the cells that contain rebar), then it is considered to be unreinforced. If the wall is grouted solid in a specified minimum number of vertical cells, then it is further investigated by NDE methods to determine if it is either reinforced or unreinforced. If the wall is found to contain enough rebar to be categorized as reinforced, it is considered to be "out-of-scope" of the evaluation guidance provided in this module. If a URM wall is determined to be a load-bearing wall, it is also considered "out-of-scope" for this module. The URM walls included in the guidance herein are assumed to be either: (1) walls that in-fill a concrete or steel frame, or (2) partitions inside a concrete or steel-framed building.

One screening approach and three methods of URM wall evaluation for out-of-plane bending are presented in this module and are the following: (1) Screening based on height/thickness ratio, (2) The Elastic Method (also called the ACI working stress approach), (3) The Reserve Energy Method, and (4) The Arching Action Method. The Elastic Method is generally the most conservative and yields a relatively low capacity for the wall in question. The Arching Action Method provides the highest capacity for the wall. Both the Reserve Energy Method and the Arching Action Method are considered to be post-elastic approaches and account for additional wall strength after wall cracking. The methods are shown in Figure 10.5.1-1.

10.5.1.1 <u>List of Selected Masonry Walls</u>

This task should be performed by others before the Seismic Capability Engineers (SCEs) begin their URM wall evaluation. A list of selected masonry walls must be generated so that the SCEs can begin their evaluation of walls. The Seismic Equipment List (SEL) is discussed in Chapter 4. If masonry walls are included on the SEL, use that list.

Questions that should be addressed during the selection of masonry walls might include:

- Is seismic interaction credible?
- Is critical equipment in the vicinity of or attached to the masonry wall?
- Is the masonry wall in question used for:
    - confinement of hazardous material?
    - shielding?
    - fire protection?
    - security concerns?

A more detailed list of questions to be addressed can be found in Reference 117, Pages 18-21.

10.5.1.2 Type of Unreinforced Masonry Wall

The three main types of masonry walls considered are:

- Concrete Masonry Unit (CMU)
- Hollow-Clay Tile (HCT)
- Brick

It will also make a difference whether each cell of the wall is grouted solid or left hollow. The hollow cell of masonry block will attract a smaller seismic loading, since it has less mass than the cell of masonry block which is fully grouted. If construction documents or installation records are not available, one must perform a non-destructive evaluation to determine the condition of the selected masonry wall. For determination of hollow cell vs. grouted cell, drilling a small hole through the face of the cell is one simple method. To ascertain whether only a few cells are grouted, check several consecutive blocks along a course of the selected wall. In some parts of the United States, insulation is placed in ungrouted cells of masonry walls. The weight of this insulation should be included when conducting the evaluations presented in this section.

It is also important to find out if the masonry wall is reinforced. The scope of the guidance in this section only includes unreinforced masonry walls. For detection of rebar, a hand-held ferromagnetic detector with a display meter or an audio signal can be easily used in many cases. An alternate method involves using imaging impulse radar. With either method, it is important to locate the positions of the following:

- vertical reinforcing steel and its approximate spacing
- horizontal reinforcing steel and its approximate spacing

An unreinforced masonry wall is a masonry wall in which the area of reinforcing steel is less than 25 percent of the minimum steel ratios required by the 1994 Uniform Building Code (UBC) for reinforced masonry (Ref. 69). Lightly or poorly reinforced walls are considered to be URM walls and can be evaluated by the methods presented in this Section.

10.5.1.3 Determine Physical Condition of Wall

As part of the seismic evaluation of the selected URM wall, it is important to examine the condition of mortar joints, openings, and existing cracks. If the mortar joints are not sound or if there are substantial cracks in the mortar or faces of the masonry units, the Elastic Method (ACI Working Stress Approach) in Section 10.5.1.5 may not be applicable.

The top connection is often not fully grouted and thus may be a free joint. Simple supports at the top and/or side should result from structural-steel angle "keepers" or dovetail slots in columns or overhead beams. There needs to be some positive means of carrying the out-of-plane load from the wall panel and into the support if it is to be considered a simple support boundary condition. If not, the wall may have to be evaluated as a cantilever.

10.5.1.4 Screening Based on Height-to-Thickness Ratio

A conservative screening approach based on the Elastic Method may be used to screen out walls from further evaluation. The top of the wall must be laterally supported to use this approach, there should be a tight fit between the supporting member, or suitable restraining members should be provided to prevent lateral motion of the top of the wall.

The wall may be screened out if:

$$\left(\frac{H}{t}\right)_{actual} \le \left(\frac{H}{t}\right)_{max}$$

where:

$$\left(\frac{H}{t}\right)_{max} = \left(\frac{H}{t}\right)_{N} \frac{\alpha_D}{\sqrt{\frac{S_{A_{max}}}{g}}}$$

$\left(\frac{H}{t}\right)_{N}$ can be found in Table 10.5.1-1 as a function of actual wall thickness t

H = wall height

t = actual wall thickness

α_D = $\sqrt{150/\rho}$ or from Table 10.5.1-6

ρ = weight density of masonry in #/ft^3

$S_{A_{max}}$ = maximum spectral acceleration from 5% damped input spectra for appropriate Performance Category and location above grade in facility (see Section 5.2). Values in Table 10.5.1-2 may only be used for Performance Category 1 masonry walls at grade.

g = acceleration of gravity

Development of this screening approach is discussed in Section 10.5.1.8.

For walls that are not screened out by this process, continue with the analysis methods presented in Sections 10.5.1.5, 10.5.1.6, and 10.5.1.7.

10.5.1.5 Elastic Method

Estimate Maximum Flexural Tensile Stress in URM Wall

For the elastic method, this module makes extensive use of Reference 117. The following topics are considered in arriving at an estimate of the maximum flexural tensile stress in the URM wall:

- natural frequency prediction for a single-wythe, uncracked masonry wall,
- determine horizontal seismic acceleration,
- estimate maximum out-of-plane bending stress for a single-wythe, uncracked, masonry wall of height H and width L

Multiple-wythe masonry walls with sufficient header courses to insure composite action can also be evaluated by this procedure. Header courses are used to tie single-wythe masonry walls together.

Determine boundary conditions of the selected URM wall

To properly use the seismic guidance in this document, it is important to determine boundary conditions of the selected URM walls. Table 10.5.1-3 lists many combinations of boundary conditions, some of which include: 1) simply supported on all four edges; 2) simply supported on top and bottom, free on sides; and 3) simply supported on bottom and sides, free on top.

Cross walls will provide support to the wall sides. Using doorways as free edges may be appropriate. However, using a window as a free edge may be overly-conservative if the window is less than half of the height of the URM wall in question.

Estimate the fundamental natural frequency of the wall

Once the boundary conditions are verified, the fundamental natural frequency can be estimated as follows:

$$f = (B_f)(F)(\alpha_E)(\alpha_D)(\alpha_T)$$

- f has units of cycles per second (Hz)
- boundary condition factor, B_f for fundamental frequency calculation from Table 10.5.1-3
- frequency factor, F from Table 10.5.1-4
- elastic modulus factor, α_E from Table 10.5.1-5
- weight density factor, α_D from Table 10.5.1-6
- orthotropic behavior adjustment factor, α_T from Table 10.5.1-7
- special considerations (for cases of partial grouting, partially filled joints, and multi-wythe walls), see Table 10.5.1-8.

Estimate the spectral acceleration of the wall

If the wall is at the ground level, the site-specific 5% damped ground response spectrum can be entered with the URM wall frequency to determine the spectral acceleration for the selected wall (see Section 5.2). If the wall is at a higher elevation in the building or if it has a basement, the appropriate floor spectrum should be used when determining the spectral acceleration of the selected wall.

Estimate the maximum flexural stress in the URM wall.

With the maximum flexural tensile stress tables, the estimated maximum flexural tensile stress for the selected wall can be scaled according to the wall spectral acceleration.

$$\sigma_b = (B_s)(S)(A_H)(1/\alpha_D)^2$$

- σ_b has units of pounds per square inch
- boundary condition factor, B_s from Table 10.5.1-9

- stress factor, S from Table 10.5.1-10
- horizontal seismic acceleration, A_H (in g's)
- weight density factor, α_D from Table 10.5.1-6.

Capacity by Elastic Method

Compare the allowable stress, due to out-of-plane seismic loads, at mortar/masonry unit interface with the estimated maximum flexural tensile stress above.

When evaluating URM walls using the Elastic Method, the following should be considered:

1. ACI 530 Table 6.3.1.1 (Ref. 118) has conservative values of allowable flexural tensile stress. Only URM walls that are located in geographic regions with low values of seismic acceleration will meet these ACI 530 code values of allowable stress.

2. The location of maximum stress depends on the specific masonry wall boundary conditions. For example, the maximum moment and stresses in many cases will occur at the fixed boundary in the form of a negative moment. In-filled walls with simple supports at the edges will most likely have the maximum out-of-plane bending stress located near the center of the wall (approximately mid-height and mid-span).

3. Values that may be used for allowable flexural stress for good quality masonry, as stated in Ref. 117, are the following:

   - 33 psi for hollow masonry
   - 52 psi for solid or fully grouted masonry

4. If site-specific test data exist, a safety factor of 2 to 3 against measured flexural tensile stress at fracture should be applied to the test results and the safety factor chosen should be consistent with the scatter of the site-specific data (Ref. 117).

Example problems illustrating application of this method are shown in Section 10.5.1.10.

10.5.1.6 Reserve Energy Method

The formulas for screening non bearing unreinforced masonry walls are developed from the arching action method with the initial confining force at the top of the wall taken as zero, (Reference 119 and 120).

For the two rigid block rocking (see Figure 10.5.1-2), the spectral acceleration capacity, S_{AP}, is

$$\frac{S_{AP}}{g} = 6\,\phi\,\frac{b}{H}\left(1 - \frac{\delta_H}{2b}\right)$$

For the cantilever wall (see Figure 10.5.1-3), the spectral acceleration capacity is

$$\frac{S_{AP}}{g} = 2\phi \frac{b}{H}\left(1 - \frac{\delta_H}{2b}\right)$$

where:

g	=	acceleration of gravity
ϕ	=	capacity reduction factor (may be taken as 0.67)
t	=	actual wall thickness
b	=	effective wall thickness = 0.9t
H	=	wall height
δ_H	=	any specified out-of-plane displacement (δ_H should be limited to no more than b for wall stability)

The Spectral Acceleration Demand, S_{AD}, can be determined by the average of the 5% damped, peak-broadened floor spectra for the floors above and below the wall at the effective frequency, f_e (see Section 5.2).

$$f_e = \frac{1}{2\pi}\sqrt{\frac{1.5\left(\frac{S_{AP}}{g}\right)g}{\delta_H}} \quad \text{(Hz)}$$

If $\quad \frac{S_{AP}}{g} \geq \frac{S_{AD}}{g}$, then the wall is acceptable.

If the capacity is less than the demand for all values of δ_H from 0 to b, the wall becomes an outlier. Wall displacement is the lowest δ_H at which $S_{AP} = S_{AD}$.

The capacity trend using the Reserve Energy Method is shown in Figure 10.5.1-4. It can be seen that the ultimate capacity S_{AP} occurs at low lateral displacement. However, the demand S_{AD} is also likely to reduce at even a faster rate with increasing δ_H (see example problems) so that the largest ratio of (S_{AP} / S_{AD}) is most likely to occur when δ_H equals the stability limit b = 0.9t.

When evaluating URM walls using the Reserve Energy Method, the following should be considered:

1. Neglect cracking strength of the unreinforced masonry wall.
2. Assume an idealized rigid-body motion of the wall.
3. Assume that the URM wall is a non-load bearing wall. Load bearing walls can also be assessed by a more complex version of the Reserve Energy Method.
4. Failure of a URM wall is identified when the response exceeds the effective wall thickness b.

Example problems illustrating application of this method are shown in Section 10.5.1.10.

10.5.1.7 Arching Action Method

Check for applicability of Arching Action. When this method can be justified, it provides the highest out-of-plane seismic capacity.

It is critical that the boundary conditions of the URM walls do not include any significant gaps (> 1/16 inch) between the top of the selected URM wall and the beam or floor above for the Arching Action Method to apply. If gaps occur, then there may be limited, or reduced, ability for the wall to develop arching action. To take credit for arching action, it is also important to check the maximum allowable compressive stress in the masonry unit and compare it to the maximum stresses developed at the edges of critical masonry units (Ref. 119).

When the rotational restraints at the boundaries are considered, a higher capacity can be achieved for the URM wall. The rotational restraint due to the wall's horizontal displacement induces an arching mechanism (Ref. 119). This arching mechanism is illustrated in Figure 10.5.1-2.

Assuming rigid body rocking develops after the masonry wall has cracked at a location αH above the base, as shown in Figure 10.5.1-2, the Reserve Energy method can be used to calculate the ultimate out-of-plane spectral acceleration capacity of a nonload bearing wall including arching action as:

$$\frac{S_{AP}}{g} = \phi \left(\frac{b}{H}\right)\left[2\, f_p \left(\frac{P_{R\delta}}{wH}\right)\left(1 - \frac{\delta_H}{b}\right) + 6\left(1 - \frac{\delta_H}{2b}\right)\right]$$

where:

g	=	acceleration of gravity
ϕ	=	capacity reduction factor (may be taken as 0.67)
t	=	actual wall thickness
b	=	effective wall thickness = 0.9t
H	=	wall height
f_p	=	$1.03 + 3.0\left(\frac{e}{b} + 0.5\right)^{0.65}$
e	=	eccentricity of P_R (see Figure 10.5.1-2)
w	=	weight/unit area of masonry wall
δ_H	=	any specified out-of-plane displacement. To take credit for arching action, δ_H should not exceed δ_p

δ_p = out-of-plane displacement at which ultimate capacity is reached $= \dfrac{0.00045H^2}{f_D t}$

except $\dfrac{\delta_p}{b} \leq \dfrac{2 F_e}{(3 - F_e)}$

f_D = 1.0 for concrete block and single wythe hollow clay tile walls
1.5 for double wythe hollow clay tile walls

$F_e = \dfrac{e}{b} + 0.5$

$P_{R\delta}$ = confining force at displacement δ_H
(increases with displacement until the displacement δ_p is reached at which the ultimate capacity occurs)

$P_{R\delta} = P_c \; f_R$

P_c = crushing capacity of block = $0.125 \; t \; f_m'$

f_m' = ultimate compressive strength of masonry
[analogus to ultimate compressive strength of concrete, f_c', typically 1000 - 1500 psi for concrete block (1350 psi typical), possibly as low as 275 psi for hollow clay tile]

f_R = relative boundary element flexibility factor (See Section 10.5.1.9 for approach used to compute f_R)
f_R should not exceed $\left(1 - \dfrac{wH}{P_c}\right)$.

The first term of the arching action capacity equation, shown above, defines the arching effect and generally dominates. For walls with large H/t and small boundary stiffness (low f_R) the second term can become very significant.

Instability will occur when δ_H reaches 0.9t. If δ_H substantially exceeds δ_p, the wall should be assumed to have lost its in-plane capacity.

The increase in capacity over the Reserve Energy Method is shown in Figure 10.5.1-5.

The effective frequency f_e is:

$$f_e = \frac{1}{2\pi}\sqrt{\frac{1.5\left(\frac{S_{AP}}{g}\right) g}{\delta_H}}$$

The spectral acceleration demand, S_{AD} can be determined from the average of the 5% damped, peak-broadened floor spectra (see Section 5.2) for the floors above and below the wall at the effective frequency f_e.

In order to determine δ_H for a given input response spectrum, start with a low δ_H and compute S_{AP}, f_e, and S_{AD}. Keep increasing δ_H until the spectral acceleration demand S_{AD} at f_e drops below the spectral acceleration capacity S_{AP} corresponding to δ_H. The lowest δ_H at which $S_{AP} \leq S_{AD}$ represents the appropriate δ_H for the given input response spectrum.

When δ_H reaches δ_p, the masonry is assumed to crush sufficiently that arching benefit is lost. For larger δ_H up to 0.9t, the capacity may be conservatively estimated by the Reserve Energy Approach discussed in the previous subsection.

The ground motion level at which the wall is acceptable can be generally established by the larger of:

1. Elastic Method Capacity
2. Reserve Energy Method Capacity with $\delta_H = b = 0.9t$
3. Arching Method Capacity with $\delta_H = \delta_p$

It is always conservative to use the larger of these three capacities. In some cases, a greater (S_{AP} / S_{AD}) ratio might occur at lesser δ_H values than the values defined above. However, in most cases, this increase is not sufficiently significant to warrant considering these intermediate δ_H values unless it is desired to have an estimate of the wall displacement for a given input spectrum.

Example problems illustration application of this method are in Section 10.5.1.10.

10.5.1.8 <u>Development of Screening Approach Based on Elastic Method</u>

A conservative screening approach has been developed to rapidly screen out walls from further analysis if they meet the screening criteria. This approach is based on the Elastic Method for walls simply supported top and bottom and free on both sides. The equations and terms used are those defined in subsection 10.5.1.5.

$$\sigma_b = B_s \, S \, A_H \left(\frac{1}{\alpha_D} \right)^2$$

$A_H = S_{A_{max}}$ = Peak of the 5% damped response spectra for the site and Performance Category, (in g's).

Use the peak of the in-structure spectra if wall is not located at grade.

$B_s = 0.125$ for walls simply supported top and bottom and free on the side.

$\alpha_D = \sqrt{150/\rho}$ from Table 10.5.1-6

$\sigma_b =$ 33 psi for hollow masonry and 52 psi for solid masonry

Therefore,

For hollow masonry:

$$S = \frac{\sigma_b \, \alpha_D{}^2}{B_s \, A_H} = \frac{33 \, \alpha_D{}^2}{0.125 \, S_{A_{max}}} = \frac{264 \, \alpha_D{}^2}{S_{A_{max}}}$$

or for solid masonry:

$$S = \frac{52 \, \alpha_D{}^2}{0.125 \, S_{A_{max}}} = \frac{416 \, \alpha_D{}^2}{S_{A_{max}}}$$

and for solid masonry:

$$S = H^2 \, w \, \frac{c}{I'} \text{ from Table 10.5.1-10}$$

$$w = \rho t$$

$$c = \frac{t}{2}$$

$$I' = \frac{t^3}{12}$$

$$S = \frac{H^2 \, \rho \, t \left(\frac{t}{2}\right)}{\frac{t^3}{12}}$$

Therefore $S = 6 \, \rho \, t \left(\frac{H}{t}\right)^2$

For hollow masonry, actual values for w and I' must be used.

Set

$$\frac{264 \, \alpha_D{}^2}{S_{A_{max}}} = H^2 \, w_{hollow} \, \frac{c}{I'_{hollow}}$$

where w_{hollow} and I'_{hollow} are the actual values for hollow masonry used to develop stress factors, S, in Table 10.5.1-10 or for solid masonry

$$\frac{416\,\alpha_D^{\,2}}{S_{A_{max}}} = 6\,\rho\,t\left(\frac{H}{t}\right)^2$$

and determine $\left(\frac{H}{t}\right)$ from the smaller value. This becomes the developed values of $\left(\frac{H}{t}\right)_N$ presented in Table 10.5.1-1.

10.5.1.9 <u>Method of Calculating Boundary Member Flexibility Factor</u> f_R

The average value of P_R along the length of the top beam can be approximated as shown in Figure 10.5.1-7. The load on the beam reaches the local block crushing capacity P_c over length *a* at each end of the beam, and is zero over the central region of the beam.

The length *a* is from the end of the beam to point 1 of Figure 10.5.1-7 at which the upward displacement δ_1 reaches

$$\delta_1 \;=\; \delta_u - \delta_g$$

where δ_g = height of any pre-existing gap between the beam and the top of the wall.

(Recall Arching Action may provide limited additional capacity if $\delta_g > \frac{1}{16}$ in.)

Vertical displacement of a simply supported beam restrained against twisting due to arching of wall is:

$$\delta_1 \;=\; \underbrace{\frac{P_c\,L^4}{32\,E I_B}\,f_R^3\left[1-\left(\frac{7}{12}\right)f_R\right]}_{\text{Flexural Term}} + \underbrace{\frac{P_c\,e_b^2\,L^2\,f_R^2}{8\,G\,J_B}}_{\text{Torsion Term}} \qquad (*)$$

where:

P_c	=	crushing capacity of block
L	=	length of beam and wall
I_B	=	moment of inertia of beam
J_B	=	polar moment of inertia of beam
E	=	elastic modulus of beam
G	=	shear modulus of beam
f_R	=	beam flexibility factor
e_b	=	eccentricity to load from beam centerline

Vertical displacement of wall due to horizontal displacement is calculated next.

As the wall blocks rock, the point at which P_R is applied lifts and presses against the boundary beam. This wall uplift at the location of P_R is given by:

$$\delta_u \quad = \quad \delta_H \left(\frac{b}{H}\right) f_p$$

Uplift Factor

$$f_p \quad = \quad 1.03 + 3.0\left(\frac{e}{b} + 0.5\right)^{0.65}$$

where e is the load eccentricity measured from the center line of the wall (see Figure 10.5.1-8), and b = 0.9t to account for block crushing.

Set vertical displacement of wall equal to vertical displacement of beam.

$$\delta_l + \delta_g = \delta_u = \delta_H \left(\frac{b}{H}\right) f_p$$

or

$$\delta_H = \left(\frac{\delta_g + \delta_l}{f_p}\right)\frac{H}{b} \qquad (**)$$

Horizontal displacement of wall at ultimate capacity

$$\delta_p = \frac{.00045\, L^2}{t} \qquad\qquad \frac{\delta_p}{b} \le \frac{2\, F_e}{3-F_e}$$

$$\delta_H \le \delta_p$$

The value of f_R can then be found by trial and error until the maximum permissible value of f_R is reached.

The following procedure can be used:

Pick f_R, start low $f_R = 0.1$, calculate δ_l from (*) on the previous page, calculate δ_H from (**) above and repeat until $\delta_H = \delta_p$

A tabular form is convenient

f_R	δ_l	δ_H
		stop when $\delta_H = \delta_p$

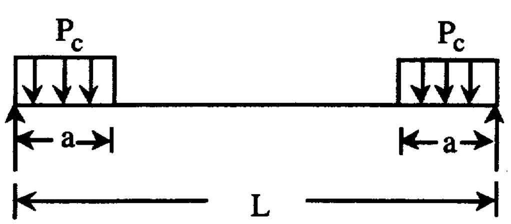
The following data will assist the calculation

$$C = f_R^3 \left[1 - \left(\frac{7}{12}\right) f_R\right]$$

then $$\delta_1 = \frac{P_c L^4}{32\, EI_B}\, C + \frac{P_c\, e_b^2\, L^2\, f_R^2}{8\, G\, J_B}$$

f_R	C
0.	0.
.1	0.000942
.2	0.00707
.3	0.0223
.4	0.0491
.5	0.0885
.6	0.140
.7	0.203
.8	0.273
.9	0.346
1.0	0.417

The boundary member capacity must also be checked. Moment capacity M_c can place an upper limit on f_R. Torsion capacity T_c can place an upper limit on e_b.



$$a = \frac{f_R L}{2}$$

$$M = P_c\, \frac{a^2}{2} = \frac{P_c\, L^2}{8}\, f_R^2 \le M_c$$

$$f_R \le \left(\frac{8\, M_c}{P_c\, L^2}\right)^{1/2}$$

$$T = P_c\, e_b\, a = \frac{P_c\, f_R\, L}{2}\, e_b \le T_c$$

$$e_b\, f_R \le \left(\frac{2\, T_c}{P_c\, L}\right)$$

10.5.1.10 Example Problems

The following example problems are presented to demonstrate application of the methods in this section to a typical URM wall.

A 6 inch hollow concrete block wall at the Portsmouth Gaseous Diffusion Plant is evaluated by the Elastic, Reserve Energy, and Arching Action Methods using ground motion described by a Portsmouth Site Specific Spectra and a Newmark and Hall Generic Spectra (Ref. 72) for a soil site.

6" Concrete Block Wall

$$f'_m = 1000 \text{ psi}$$

$$H = 12' = 144"$$

$$L = 18' = 216"$$

$$\rho = 135 \text{ lbs/ft}^3$$

Simply supported top and bottom, free on sides

Portsmouth Site with 0.15g spectrum (see Figure 10.5.1-6A)

Screening Approach (Section 10.5.1.4)

$$\left(\frac{H}{t}\right)_{actual} = \frac{144}{5.625} = 25.6$$

$$SA_{max} = 0.4g \text{ (Portsmouth)}$$

$$SA_{max} = 2.12 \times .15 = 0.32g \text{ (Newmark \& Hall)}$$

$$\alpha_D = \sqrt{\frac{150}{135}} = 1.054$$

$$\left(\frac{H}{t}\right)_{max} = \left(\frac{H}{t}\right)_N \frac{\alpha_D}{\sqrt{SA_{max}}}$$

$$\left(\frac{H}{t}\right)_N = 11.5 \text{ for a 6" wall from Table 10.5.1-1}$$

$$\left(\frac{H}{t}\right)_{max} = \frac{(11.5)\,(1.054)}{\sqrt{0.4}} = 19.17 \qquad \text{(Portsmouth ground motion)}$$

$$\left(\frac{H}{t}\right)_{max} = \frac{(11.5)\,(1.054)}{\sqrt{0.32}} = 21.43 \qquad \text{(Newmark and Hall ground motion)}$$

$$\left(\frac{H}{t}\right)_{actual} > \left(\frac{H}{t}\right)_{max}$$

Wall is not screened out.

Elastic Method (Section 10.5.1.5)

Estimate seismic capacity from:

$$\sigma_b = B_s\ S\ A_H \left(\frac{1}{\alpha_D}\right)^2$$

$$\sigma_b = \sigma_b \text{ allowable} = 33 \text{ psi}$$

$$\frac{H}{L} = \frac{12'}{18'} = 0.67, \ B_s = 0.125 \text{ from Table 10.5.1-9}$$

$$S = 1245 \text{ psi from Table 10.5.1-10}$$

$$\alpha_D = \sqrt{\frac{150}{135}} = 1.054$$

$$A_H = S_{AP} = \frac{\sigma_b\ \alpha_D^2}{B_S S} = \frac{(33)\,(1.054)^2}{(0.125)\,(1245)} = 0.24g$$

Estimate frequency from:

$$f = B_f\ F\ \alpha_E\ \alpha_D\ \alpha_T$$

$$\frac{H}{L} = 0.67, \ B_f = 1.571 \text{ from Table 10.5.1-3}$$

6" hollow concrete block, H = 12', F = 6.70 from Table 10.5.1-4

$$\alpha_E = 1 \text{ from Table 10.5.1-5}$$

$$\alpha_D = 1.054$$

$$\alpha_T = 0.97 \text{ from Table 10.5.1-7 for 6" wall}$$

$$f = (1.571)(6.70)(1)(1.054)(0.97) = 10.8 \text{ Hz}$$

$$T = \frac{1}{f} = 0.093 \text{sec}$$

S_{AD} = 0.4g from 0.15g Portsmouth 5% damped spectra at 0.093 sec

Capacity to Demand Ratio = $\dfrac{S_{AP}}{S_{AD}} = \dfrac{0.24}{0.40} = 0.6 < 1.0$

Wall Fails Elastically

The maximum elastic peak ground acceleration that will not fail the wall elastically is

$$a_g = (0.60)(0.15g) = 0.09g$$

<u>Reserve Energy Method (Section 10.5.1.6)</u>

$b = 0.9t = 0.9\ (6") = 5.4"$
(Note: 6" is the nominal wall thickness, the actual wall thickness should be used in the calculation).

$$\frac{S_{AP}}{g} = 6\phi\frac{b}{H}\left(1-\frac{\delta_H}{2b}\right)$$

$$\frac{S_{AP}}{g} = 6\,(0.67)\left(\frac{5.4}{144}\right)\left[1 - \frac{\delta_H}{2(5.4)}\right]$$

$$\frac{S_{AP}}{g} = 0.151\left[1 - \frac{\delta_H}{10.8}\right]$$

$$f_e = \frac{1}{2\pi}\sqrt{\frac{1.5\,S_{AP}\,g}{\delta_H}} = \frac{1}{2\pi}\sqrt{\frac{(1.5)\,S_{AP}\,(386.4)}{\delta_H}} = 3.83\left(\frac{S_{AP}}{\delta_H}\right)^{0.5}\ \text{Hz}$$

Find S_{AP}, f_e, and S_{AD} at various δ_H up to stability limit of 5.4".

Reserve Energy Results in tabular form:

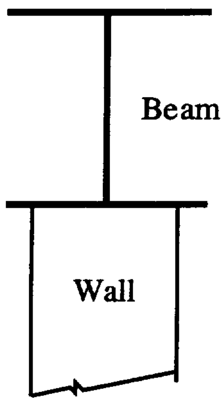
δ_H (inch)	Capacity S_{AP} (g)	Frequency f_e (Hz)	Period T (sec)	Demand S_{AD} (g)	Capacity/ Demand $\frac{S_{AP}}{S_{AD}}$	a_g (g)
0.2	.148	3.29	0.30	.215	0.69	0.10
0.4	.145	2.31	0.43	.145	1.00	0.15(1)
1.0	.137	1.42	0.70	.066	2.08	0.31
2.0	.123	.95	1.05	.036	3.42	0.51
5.4	.076	.45	2.22	.012	6.33	0.95(2)

1. Wall displaces only 0.4" for 0.15g Spectrum
2. Wall reaches stability limit at 0.95g Spectrum

Much greater capacity than for Elastic Method because spectrum drops quickly at lower frequencies.

Arching Action Method (Section 10.5.1.7) - Case 1

Case 1: Simply supported steel W8 x 28 beam centered on top of wall with no gap between beam and top of wall



Negligible torsional resistance
web of beam lines up with
centerline of wall, $e_o = 0$ (see Figure 10.5.1-8)

Use $e = 0$

$e_b = 0$

$E = 29 \times 10^6$ psi

$I_B = 98$ in.^{4}

$L = 216$ in.

Masonry:

$f'_m = 1000$ psi

$P_c = .125 t f'_m = .125\,(6")\,(1000\,\text{psi}) = 750 \,\#/\text{in.}$

$w = \rho t = (135)\,(0.5)\,\#/\text{ft}^2 = 0.469$ psi

gap $= \delta_g = 0$

Vertical displacement of beam:

$$\delta_1 = \frac{P_c L^4}{32 E I_B} f_R^3 \left\{1 - \left(\frac{7}{12}\right) f_R\right\} = \frac{750\,\#/\text{in.}\,(216\,\text{in.})^4}{32\left(29 \times 10^6\,\text{psi}\right)\left(98\,\text{in}^4\right)} f_R^3\,(1 - .583\,f_R)$$

$$\delta_1 = 17.95"\ f_R^3(1 - .583\,f_R) = 17.95\ C$$

Displacement at ultimate capacity:

$$\delta_p = \frac{.00045(144")^2}{(6")} = 1.56"$$

Set $\delta_H \le \delta_p = 1.56"$

Uplift factor:

$$f_p = 1.03 + 3.0\left(\frac{e}{b} + 0.5\right)^{0.65} = 1.03 + 3.0\,(.5)^{.65} = 2.94$$

$$\delta_u = \delta_H\left(\frac{b}{H}\right) f_p = 2.94\left(\frac{5.4}{144}\right)\delta_H = 0.110\ \delta_H$$

$$\delta_u - \delta_g = \delta_1$$

$$\delta_u = \delta_1 = 0.110\ \delta_H$$

$$\delta_H = \frac{\delta_1}{0.110}$$

Maximum permissible f_R:

$$f_R \le \left(1 - \frac{wH}{P_c}\right) \le 0.91$$

Check steel W8 x 28 beam A36 steel:

$$M_{CAP} = \phi\ F_y\ Z_x = (0.9)\,(36\,\text{ksi})\left(27.2\,\text{in.}^3\right) = 881\ \text{k-in} \qquad \text{(LRFD Method)}$$

$$f_R \le \left[\frac{8\,(881)}{.750\,(216)^2}\right]^{\frac{1}{2}} = 0.45$$

thus $f_R \le 0.45$

$T_{CAP} \approx 0$ for wide flange held only on web at ends

$$e_b = 0$$

$$e = e_b - e_o = 0 - 0 = 0$$

Start by picking a $f_R = 0.10$, calculate δ_1 and δ_H until $\delta_H = \delta_p = 1.56$:

f_R	δ_1 (in)	δ_H (in)
0.10	.0169	.154
0.14	.0452	.411
0.20	.127	1.15
0.22	.167	1.52
0.225	.178	1.61

← max δ_H for arching (block begins to crush) $f_R \approx 0.222$

$$P_{R\delta} = P_c f_R$$

$$\frac{S_{AP}}{g} = \phi\left(\frac{b}{H}\right)\left[2 f_p \frac{P_{R\delta}\left(1-\frac{\delta_H}{b}\right)}{wH} + 6\left(1-\frac{\delta_H}{2b}\right)\right]$$

$$\frac{S_{AP}}{g} = 0.67\left(\frac{5.4}{144}\right)\left[2(2.94)\frac{750\frac{\#}{''} f_R\left(1-\frac{\delta_H}{5.4''}\right)}{.469\,\text{psi}\,(144'')} + 6\left(1-\frac{\delta_H}{10.8}\right)\right]$$

$$\frac{S_{AP}}{g} = \underbrace{1.64 f_R\left(1-\frac{\delta_H}{5.4}\right)}_{\substack{\text{Arching} \\ \text{(only good up to 1.56")}}} + \underbrace{0.151\left(1-\frac{\delta_H}{10.8}\right)}_{\substack{\text{Reserve} \\ \text{Energy}}}$$

$$f_e = 3.83\left(\frac{S_{AP}}{\delta_H}\right)^{0.5}$$

Arching Action results:

δ_H (inch)	Capacity S_{AP} (g)	Frequency f_e (Hz)	Period T (sec)	Demand S_{AD} (g)	$\frac{S_{AP}}{S_{AD}}$	a_g (g)	
0.154	0.300	5.35	.187	.342	0.88	0.13	
0.200					1.00	0.15	
0.411	0.357	3.57	.280	.228	1.57	0.24	Arching Action
1.15	0.393	2.24	.446	.129	3.05	0.46	
1.56	0.388	1.91	.524	.101	3.84	0.58	
2.0	0.123	.95	1.05	.036	3.42	0.51	Reserve Energy
5.4	0.076	.45	2.22	.012	6.33	0.95	

Wall displaces only 0.2" for 0.15g Spectrum (by interpolation) (Only about 50% of Reserve Energy deflection)

Stability limit is still 0.95g Spectrum (Same as for Reserve Energy)

Not much benefit from arching because of flexibility of support beam and quick drop-off with lowering frequency for input spectrum.

Arching Action Method (Section 10.5.1.7) - Case 2

Case 2: Same wall, but supported by a large simply supported, torsionally restrained reinforced concrete beam with the following properties:

$I_B = 6000 \text{ in}^4$ $\qquad E = 3 \times 10^6 \text{ psi}$

$J_B = 7000 \text{ in}^4$ $\qquad G = 1.2 \times 10^6 \text{ psi}$

see Figure 10.5.1-8

$e_o = 0$ $\qquad e/b = 0.5$ $\qquad e_b = \frac{b}{2} - e_o = 0.45t - 0 = 2.7"$

$$f_p = 1.03 + 3.0\,(1.0)^{.65} = 4.03$$

$$\delta_u - \delta_g = \delta_1 = \delta_H \left(\frac{5.4"}{144"}\right)(4.03) = 0.151\ \delta_H$$

$$\delta_H = \frac{\delta_1}{0.151}$$

$$\delta_1 = \frac{P_c L^4}{32\, EI_B} f_R^3 \left\{1 - \frac{7}{12} f_R\right\} + \frac{P_c e_b^2 L^2 f_R^2}{8\, GJ_B}$$

$$\delta_1 = \underbrace{\frac{750\#/" (216")^4}{32\left(3\times 10^6 \text{ psi}\right)\left(6000"^4\right)} f_R^3 \left(1 - .583 f_R\right)}_{\text{Flexure}} + \underbrace{\frac{750\#/" (2.7")^2 (216")^2}{8\left(1.2 \times 10^6 \text{ psi}\right)\left(7000"^4\right)} f_R^2}_{\text{Torsion}}$$

$$\delta_1 = 2.83" f_R^3 \left(1 - .583 f_R\right) + (\approx 0)$$

Maximum permissible f_R:

$$f_R \le \left(1 - \frac{wH}{P_c}\right) = 1 - \frac{.469 \text{ psi } (144")}{750\#/"} = 0.91$$

Check concrete beam $\left(12" \times 24" \text{ Deep}, A_s \ge 2 \text{ in}^2\right)$ with some torsional steel:

$$M_{CAP} = 2000 \text{ k-in.}$$

$$f_R \le \left[\frac{8\,(2000)}{.750\,(216)^2}\right]^{\frac{1}{2}} = 0.68$$

thus $f_R \le 0.68$

$$T_{CAP} = 120 \text{ k-in.}$$

$$e_b\, f_R \le \left[\frac{2\,(120)}{.750\,(216)}\right] = 1.48$$

e_b must be reduced below 2.7" if

f_R exceeds $\frac{1.48}{2.7} = 0.55$

f_R	δ_1 (in.)	$\delta_H = \frac{\delta_1}{0.151}$ (in.)
0.20	.0200	0.132
0.25	.0378	0.250
0.30	.0630	0.417
0.40	.139	0.920
0.486		1.56
0.50	.251	1.66

$$\frac{S_{AP}}{g} = 2.25\, f_R \left(1 - \frac{\delta_H}{5.4}\right) + 0.151\left(1 - \frac{\delta_H}{10.8}\right)$$

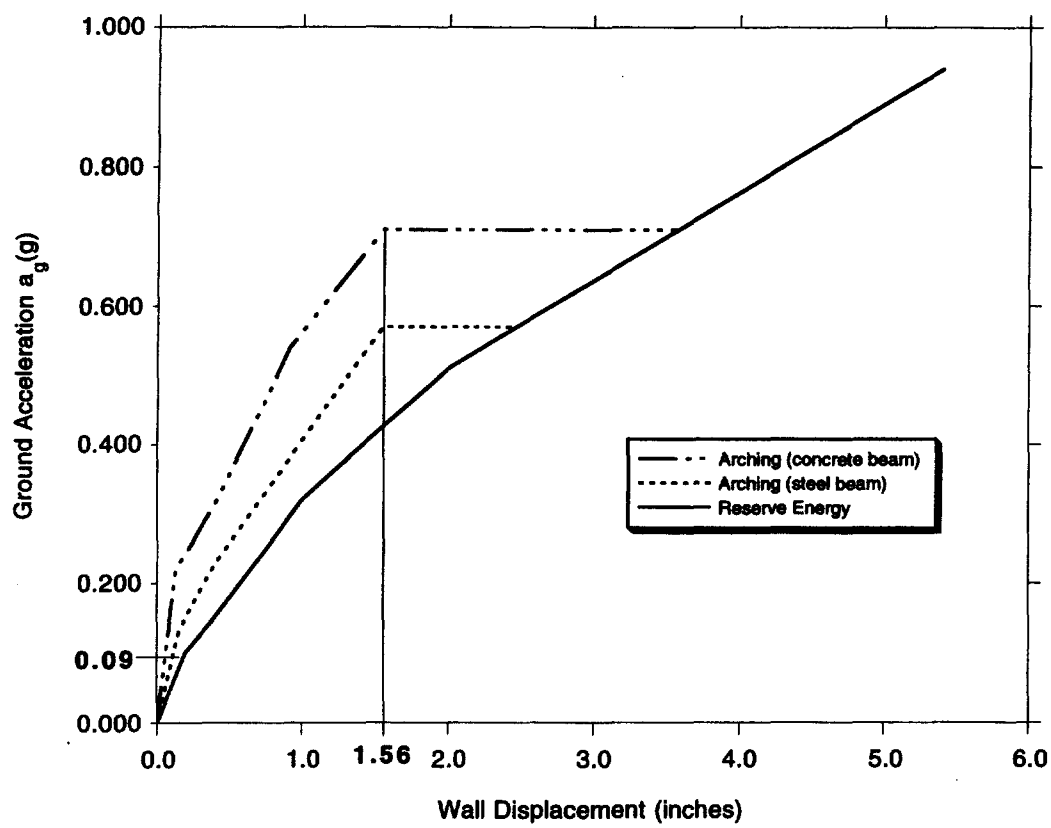
$$f_e = 3.83\sqrt{\frac{S_{AP}}{\delta_H}}$$

δ_H (inch)	S_{AP} (g)	f_e (Hz)	T (sec)	S_{AD} (g)	$\frac{S_{AP}}{S_{AD}}$	a_g (g)	
0.132	.588	8.08	0.124	0.40	1.47	0.22	
0.250	.684	6.33	0.158	0.40	1.71	0.26	
0.417	.768	5.20	0.192	0.338	2.27	0.34	Arching Action
0.920	.885	3.76	0.266	0.245	3.61	0.54	
1.56	.907	2.92	0.342	0.191	4.75	0.71	
2.0	.123	0.95	1.05	.036	3.42	0.51	Reserve Energy
5.4	.076	0.45	2.22	.012	6.33	0.95	

Wall displays only 0.13 inches for a 0.22g input
However, stability limit is still 0.95g

Arching Action did not increase stability limit because of shape of input spectrum.

Comparison of results for Portsmouth input spectrum shape:



Rework same example with NUREG/CR-0098 (Ref. 72) input median spectrum for a soil site to illustrate the importance of the input spectrum shape on relative results.

Spectrum properties for 5% damping are given below and shown in Figure 10.5.1-6B:

$8\,\text{Hz} \le f \le 33\,\text{Hz}$:	S_{AD}	$=$	$a_g \left(f/33\text{Hz}\right)^{-0.53}$
$1.64\ \text{Hz} \le f \le 8\,\text{Hz}$	S_{AD}	$=$	$2.12\ a_g$
$0.25\ \text{Hz} \le f \le 1.64\ \text{Hz}$	S_{AD}	$=$	$1.29\,\text{sec}\ f\ a_g$
$f \le 0.25\ \text{Hz}$	S_{AD}	$=$	$5.08\,\text{sec}\ f^2\ a_g$

Elastic Method (Section 10.5.1.5)

$$S_{AP} = 0.24g$$

$$f = 10.8\,\text{Hz} \longrightarrow S_{AD} = 1.81\ a_g = 0.27g$$

$$\frac{S_{AP}}{S_{AD}} = \frac{0.24}{0.27} = 0.89 < 1.0$$

$$a_g = (0.89)\,(0.15g) = 0.13g$$

Reserve Energy Method (Section 10.5.1.6)

Using previous results:

δ_H (in.)	S_{AP} (g)	f_e (Hz)	S_{AD}/a_g	$a_g = \left(\dfrac{\frac{S_{AP}}{S_{AD}}}{a_g}\right)$ (g)	
0.20	.148	3.29	2.12	0.07	less than elastic
0.40	.145	2.31	2.12	0.07	"
1.0	.137	1.42	1.83	0.07	"
2.0	.123	.95	1.23	0.10	"
5.4	.076	.45	0.58	0.13	"

No value over Elastic Method

For NUREG/CR-0098 soil spectrum, wall becomes unstable when it exceeds 0.13g elastic capacity, no advantage to Reserve Energy Method. (Spectrum has lots of low frequency)

Arching Action Method (Section 10.5.1.7) - Case 1

Case 1 - Steel beam

Using previous results:

δ_H (in.)	S_{AP} (g)	f_e (Hz)	S_{AD}/a_g	$a_g = \left(\dfrac{S_{AP}}{\frac{S_{AD}}{a_g}} \right)$ (g)
.154	.300	5.35	2.12	0.14
.411	.357	3.57	2.12	0.17
1.15	.393	2.24	2.12	0.18
1.56	.388	1.91	2.12	0.18

Maximum $a_g \approx 1.4$ * Elastic capacity for NUREG/CR-0098 soil spectrum

Arching Action Method (Section 10.5.1.7) - Case 2

Case 2 - Concrete beam

Using previous results:

δ_H (in.)	S_{AP} (g)	f_e (Hz)	S_{AD}/a_g	$a_g = \dfrac{S_{AP}}{\frac{S_{AD}}{g}}$ (g)
.132	.588	8.08	2.11	0.28
.250	.684	6.33	2.12	0.32
.417	.768	5.20	2.12	0.36
.920	.885	3.76	2.12	0.42
1.56	.907	2.92	2.12	0.43

Maximum $a_g \approx 3.3$ *Elastic capacity for NUREG/CR-0098 soil spectrum

Summary of Section 10.5.1.10

	Factor Over Elastic a_g Capacity	
	Portsmouth Spectrum	NUREG/CR-0098 Soil Spectrum
Reserve Energy	10.6	1.0
Arching Case 1 (Steel Beam)	10.6	1.4
Arching Case 2 (Concrete Beam)	10.6	3.3

Whether Reserve Energy results in increased capacity over Elastic Method is highly sensitive to shape of input demand spectrum.

Increase in capacity from Arching Action is significantly influenced by stiffness of boundary element and shape of input demand spectrum.

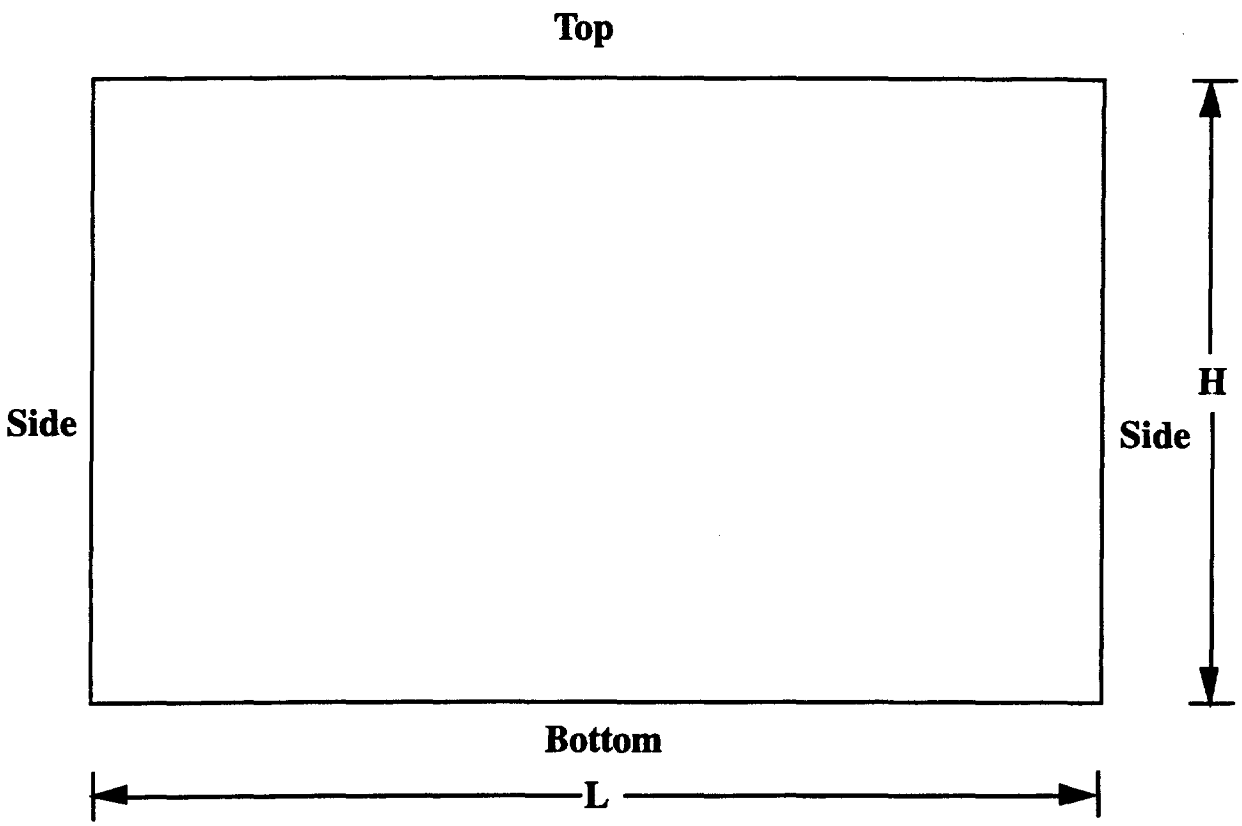
Table 10.5.1-1 $\left(\frac{H}{t}\right)_N$ **versus Wall Thickness for use in URM Wall Screening (based on Sections 10.5.1.4 and 10.5.1.8)**

Nominal URM Wall Thickness	Actual Concrete Block Wall Thickness	Minimum Concrete Block Flange Thickness	$\left(\frac{H}{t}\right)_N$
4"	3.625"	.75"	13.5
6"	5.625"	1.0"	11.5
8"	7.625"	1.25"	10.0
10"	9.625"	1.375"	9.0
12"	11.625"	1.5"	8.0

Table 10.5.1-2 DBE Ground Motion $S_{A_{max}}$ from UBC Seismic Zone (May be used for PC 1 Structures, Systems, and Components, Ref. 6)

DOE Site	Seismic Zone	$S_{A_{max}}$
Kansas City	2A	0.41
LANL	2B	0.55
Mound	1	0.21
Pantex Plant	1	0.21
Rocky Flats	1	0.21
Sandia, Albuquerque	2B	0.55
Sandia, Livermore	4	1.10
Pinellas Plant	0	0.10
Argonne-East	0	0.10
Argonne-West	2B	0.55
Brookhaven	2A	0.41
Princeton	2A	0.41
INEL	2B	0.55
Feed Materials Production Center	1	0.21
Oak Ridge	2A	0.41
Paducah	2A	0.41
Portsmouth	1	0.21
Nevada Test Site	3	0.83
Hanford	2B	0.55
LBL	4	1.10
LLNL	4	1.10
ETEC	4	1.10
SLAC	4	1.10
Savannah River	2A	0.41

Table 10.5.1-3 Boundary Condition Factors, B_F, for Fundamental Frequency Calculation (Table 1 of Reference 117)



Case 1: Simple Support Top/Simple Support Bottom with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤ 0.20	1.571	1.571	1.571	1.571	1.571	1.571
0.4	1.571	1.612	1.622	1.822	1.870	1.931
0.667	1.571	1.698	1.748	2.270	2.480	2.765
1.0	1.571	1.859	2.020	3.142	3.764	4.608
1.5	1.571	2.182	2.677	5.106	6.769	8.968
2.5	1.571	2.992	4.875	11.39	16.54	23.16

Table 10.5.1-3 (Continued)

Case 2: Fixed Top/Fixed Bottom with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤ 0.20	3.561	3.561	3.561	3.561	3.561	3.561
0.4	3.561	3.587	3.594	3.706	3.731	3.764
0.667	3.561	3.638	3.664	3.986	4.116	4.299
1.0	3.561	3.734	3.823	4.608	5.066	5.730
1.5	3.561	3.944	4.254	6.221	7.666	9.672
2.5	3.561	4.545	5.994	12.07	17.05	23.52

Case 3: Simple Support Top/Fixed Bottom (or Vice-Versa) with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤ 0.20	2.454	2.454	2.454	2.454	2.454	2.454
0.4	2.454	2.491	2.499	2.646	2.682	2.727
0.667	2.454	2.558	2.593	3.008	3.175	3.407
1.0	2.454	2.685	2.804	3.764	4.307	5.066
1.5	2.454	2.951	3.349	5.579	7.144	9.260
2.5	2.454	3.672	5.344	11.69	16.76	23.32

Table 10.5.1-3 (Continued)

Case 4: Free Top/Fixed Bottom with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤0.20	0.560	0.560	0.560	0.560	0.560	0.560
0.4	0.560	0.613	0.634	0.780	0.855	0.959
0.667	0.560	0.704	0.793	1.190	1.488	1.891
1.0	0.560	0.897	1.105	2.020	2.804	3.823
1.5	0.560	1.103	1.786	3.932	5.833	8.243
2.5	0.560	1.607	3.965	10.14	15.62	22.46

Case 5: Free Top/Simple Support Bottom with Specified Combination of Side Supports

H/L	Free-Free*	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤0.2	0	0.107	0.159	0.224	0.258	0.285
0.4	0	0.210	0.257	0.479	0.587	0.727
0.667	0	0.356	0.491	0.971	1.313	1.755
1.0	0	0.536	0.854	1.859	2.685	3.734
1.5	0	0.800	1.585	3.821	5.755	8.186
2.5	0	1.313	3.834	10.08	15.57	22.42

* Rigid Body Mode

Table 10.5.1-4 Frequency Factors, F
(Table 2 of Reference 117)

WALL HEIGHT H	HOLLOW MASONRY THICKNESS					SOLID MASONRY THICKNESS				
	4"	6"	8"	10"	12"	4"	6"	8"	10"	12"
6'	17.4	26.8	36.5	45.8	55.1	13.5	20.9	28.3	35.8	43.2
8'	9.81	15.1	20.5	25.7	31.0	7.57	11.8	15.9	20.1	24.3
10'	6.28	9.65	13.1	16.5	19.8	4.85	7.52	10.2	12.9	15.5
12'	4.36	6.70	9.13	11.4	13.8	3.37	5.22	7.08	8.94	10.8
14'	3.20	4.92	6.71	8.41	10.1	2.47	3.84	5.20	6.57	7.94
16'	2.45	3.77	5.14	6.44	7.75	1.89	2.94	3.98	5.03	6.07
18'	1.94	2.98	4.06	5.09	6.13	1.50	2.32	3.15	3.97	4.79
20'	1.57	2.41	3.29	4.12	4.96	1.21	1.88	2.55	3.22	3.88
24'	1.09	1.68	2.28	2.86	3.45	.841	1.31	1.77	2.23	2.70
30'	.698	1.07	1.46	1.83	2.21	.538	.836	1.13	1.43	1.73

$$F = (1/H^2) * (EI'g/w)^{1/2}$$

where

H = Wall Height (in)

E = Elastic Modulus = 1×10^6 #/in^2

I' = Effective Plate Moment of Inertia (in^4/in)

g = Acceleration of Gravity = 386.4 in/sec^2

w = Distributed Load per Unit Surface Area (#/in^2) based on masonry weight density = 150 #/ft^3

Table 10.5.1-5 Elastic Modulus Factor (α_E)
(Table 3 of Reference 117)

The Frequency Factor, F, is based on E = 1 x 10^6 psi. To adjust f for other values of E, $\alpha_E = \sqrt{E/(1x10^6)}$. For masonry, E is typically taken as 1000 f'_m, where f'_m is the compressive strength of the masonry unit/mortar combination. The typical range of E is 0.7 x 10^6 psi to 2.5 x 10^6 psi. Site-specific testing can be utilized to determine E.

The following table shows α_E vs. E for the range of interest:

E (psi)	α_E
0.5 x 10^6	0.71
0.7 x 10^6	0.84
0.9 x 10^6	0.95
1.0 x 10^6	1.0
1.25 x 10^6	1.12
1.50 x 10^6	1.22
1.75 x 10^6	1.32
2.00 x 10^6	1.41
2.25 x 10^6	1.50
2.50 x 10^6	1.58
2.75 x 10^6	1.66
3.00 x 10^6	1.73

Table 10.5.1-6 Weight Density Factor (α_D) (Table 4 of Reference 117)

The Frequency Factor, F, is based on a weight density, ρ, of 150#/ft^3 for the masonry material. Based on the density, the masonry block construction (solid vs. hollow), and the nominal block thickness (4", 6", 8", 10", 12"), the surface loading, w, is defined in #/in^2.

The density of masonry may vary over a wide range, depending on the application. By varying aggregate density and constituent ratios, ρ can range from 75 #/ft^3 to 200 #/ft^3. For most DOE facilities, the reference value of ρ = 150 #/ft^3 should be a suitable, slightly conservative value.

To account for cases where there is significant difference, based on site-specific design specifications or sample testing, the following table provides values of α_D vs. ρ for the expected range of variation:

ρ (#/ft.^{3})	α_D
200	0.87
175	0.93
150	1.0
125	1.10
100	1.22
75	1.41

To adjust f for other values of ρ, $\alpha_D = \sqrt{150/\rho}$

Additional Weight of Attachments

To account for the additional weight of attachments to the wall, an effective weight density can be estimated as follows:

1. Estimate total weight of attachments, WT_A

2. Divide WT_A by gross wall volume (HxLxt) to get effective increase in density

   $\rho_A = WT_A$ / (HLt) [#/ft^3]

3. For solid masonry, effective total density is

   $\rho = \rho_{masonry} + \rho_A$

4. For hollow masonry, effective total density is

   $\rho = \rho_{masonry} + 2\ (\rho_A)$

   The factor of 2 on ρ_A for hollow masonry accounts for the fact that the net volume is approximately 50% of the gross volume.

5. Select factor α_D based on the effective total density.

Table 10.5.1-7 Orthotropic Behavior Adjustment Factor (α_T) (Table 5 of Reference 117)

A. Solid Masonry

For solid masonry (including hollow masonry with completely grouted cells), isotropic out-of-plane bending behavior is expected. Consequently,

$$\alpha_T = 1.0$$

B. Hollow Masonry

Based on the geometry of the hollow masonry, the section properties resisting out-of-plane bending are different for bending about axes perpendicular to and parallel to the cell axis direction. Assuming completely mortared web joints between masonry units, the webs contribute to the bending resistance about an axis perpendicular to the cell axis direction. For bending about an axis parallel to the cell axis direction, the webs are considered to be ineffective; this results in a modest reduction of bending resistance, which is a function of the masonry unit thickness. The significance of this reduction on the out-of-plane natural frequency depends on the plate aspect ratio and the cell axis direction. The worst case reduction factors are provided in the table below for the range of masonry unit thicknesses:

Hollow Masonry Unit Thickness (in.)	α_T (minimum value)
4"	0.98
6"	0.97
8"	0.96
10"	0.94
12"	0.91

A more accurate value for α_T can be determined by the following procedure:

1) Calculate the wall aspect ratio (AR), defined as the lineal dimension parallel to the cell axis divided by the lineal dimension perpendicular to the cell axis:
2) For AR $\leq$ 0.2, use $\alpha_T = 1.0$.
3) For AR $\geq$ 5.0, use α_T (min) = 0.91.
4) For AR = 1.0, use $\alpha_T = 0.5\ [1.0 + \alpha_T\ (\text{min})]$.
5) For 0.2 < AR < 1.0, use linear interpolation between 1.0 and $0.5\ [1.0 + \alpha_T\ (\text{min})]$.
6) For 1.0 < AR < 5.0, use linear interpolation between $0.5\ [1.0 + \alpha_T\ (\text{min})]$ and α_T (min).

Table 10.5.1-8 Special Considerations for Elastic Method (Table 6 of Reference 117)

A) Partial Grouting of Cells in Hollow Masonry

If selected cells are grouted from top to bottom of the wall, in a regular pattern, then both wall mass and stiffness are increased. This would tend to decrease the applicable frequency factor, F. Therefore, the solid masonry values in Table 10.5.1-4 can be used as a conservative lower bound for F. Alternately, interpolation between the solid and hollow masonry values can be used, based on the percentage of cells filled.

B) Partially Filled Mortar Joints

1) Solid Masonry

This is an undesirable condition, which raises questions about the original construction workmanship. A technical basis for such construction should be investigated. In addition, a significant amount of in-situ sampling is probably required to characterize the mortar joints

2) Hollow Masonry

The original construction may not have specified mortaring of the webs in the bed joints. If this condition has been verified by in-situ sampling then the Orthotropic Behavior Adjustment Factor, α_T, is set to the appropriate minimum value from Table 10.5.1-5 in the calculation of the wall frequency. This effectively eliminates any contribution to bending stiffness from the webs.

Any other deviation from fully mortared joints is an undesirable condition. Refer to discussion above for solid masonry.

C) Multi-Wythe and Composite Construction

The possible combinations are too numerous to quantify. However, certain guidance can be provided for the assessment of such walls.

1) If adequate connectivity between wythes cannot be demonstrated, then each wythe must be treated as a separate wall. In this case, the formulas and data provided here should be applicable to each wythe.

2) Adequate connectivity should be verified by definitive design and fabrication documentation, supported by in-situ sampling.

3) The Boundary Condition Factor, B_f from Table 10.5.1-3 is applicable to multi-wythe and composite construction. A case-specific Frequency Factor, F, would have to be developed for composite bending behavior.

Table 10.5.1-9 Boundary Condition Factors, B_s, for Maximum Bending Stress Calculation (Table 7 of Reference 117)

Case 1: SS Top/SS Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.125	0.125	0.125
0.4	0.125	0.110	0.122
0.667	0.125	0.081	0.105
1.0	0.125	0.048	0.070
1.5	0.125	0.036	0.037
2.5	0.125	0.018	0.013

Case 2: Fixed Top/Fixed Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.083	0.083	0. 083
0.4	0.083	0.083	0.083
0.667	0.083	0.082	0.076
1.0	0.083	0.070	0.051
1.5	0.083	0.047	0.034
2.5	0.083	0.020	0.013

Table 10.5.1-9 (Continued)

Case 3: SS Top/Fixed Bottom (or Vice-Versa)

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.125	0.125	0.125
0.4	0.125	0.125	0.119
0.667	0.125	0.110	0.095
1.0	0.125	0.084	0.060
1.5	0.125	0.050	0.034
2.5	0.125	0.020	0.013

Case 4: Free Top/Fixed Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.50	0.50	0.50
0.4	0.50	0.375	0.275
0.667	0.50	0.227	0.173
1.0	0.50	0.119	0.085
1.5	0.50	0.055	0.037
2.5	0.50	0.021	0.013

Table 10.5.1-9 (Continued)

Case 5: Free Top/Simple Support Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
0.2	*	0.78	0.78
0.4	*	0.34	0.34
0.667	*	0.187	0.187
1.0	*	0.112	0.085
1.5	*	0.057	0.037
2.5	*	0.021	0.013

* Unstable Condition

Table 10.5.1-10 Stress Factors, S (psi)
(Table 8 of Reference 117)

WALL HEIGHT H	HOLLOW MASONRY THICKNESS					SOLID MASONRY THICKNESS				
	4"	6"	8"	10"	12"	4"	6"	8"	10"	12"
6'	460	310	230	195	170	745	480	355	280	230
8'	815	555	410	345	305	1,325	850	630	500	415
10'	1,275	865	640	545	475	2,075	1,330	985	780	645
12'	1,835	1,245	925	780	680	2,985	1,915	1,415	1,120	930
14'	2,500	1,695	1,255	1,065	930	4,065	2,610	1,930	1,525	1,265
16'	3,260	2,215	1,640	1,390	1,215	5,310	3,405	2,520	1,995	1,650
18'	4,130	2,805	2,075	1,760	1,535	6,720	4,310	3,185	2,525	2,090
20'	5,100	3,460	2,565	2,170	1,895	8,295	5,320	3,935	3,115	2,580
24'	7,340	4,985	3,690	3,125	2,730	11,945	7,665	5,665	4,485	3,715
30'	11,470	7,790	5,765	4,885	4,265	18,660	11,975	8,850	7,010	5,805

$S = H^2 * (wc/I')$

where
- H = Wall Height (in)
- I' = Effective Plate Moment of Inertia (in^4/in)
- c = Distance from Neutral Axis to Extreme Fiber (in)
- w = Distributed Load per Unit Surface Area ($\#/in^2$) based on masonry weight density = $150\#/ft^3$

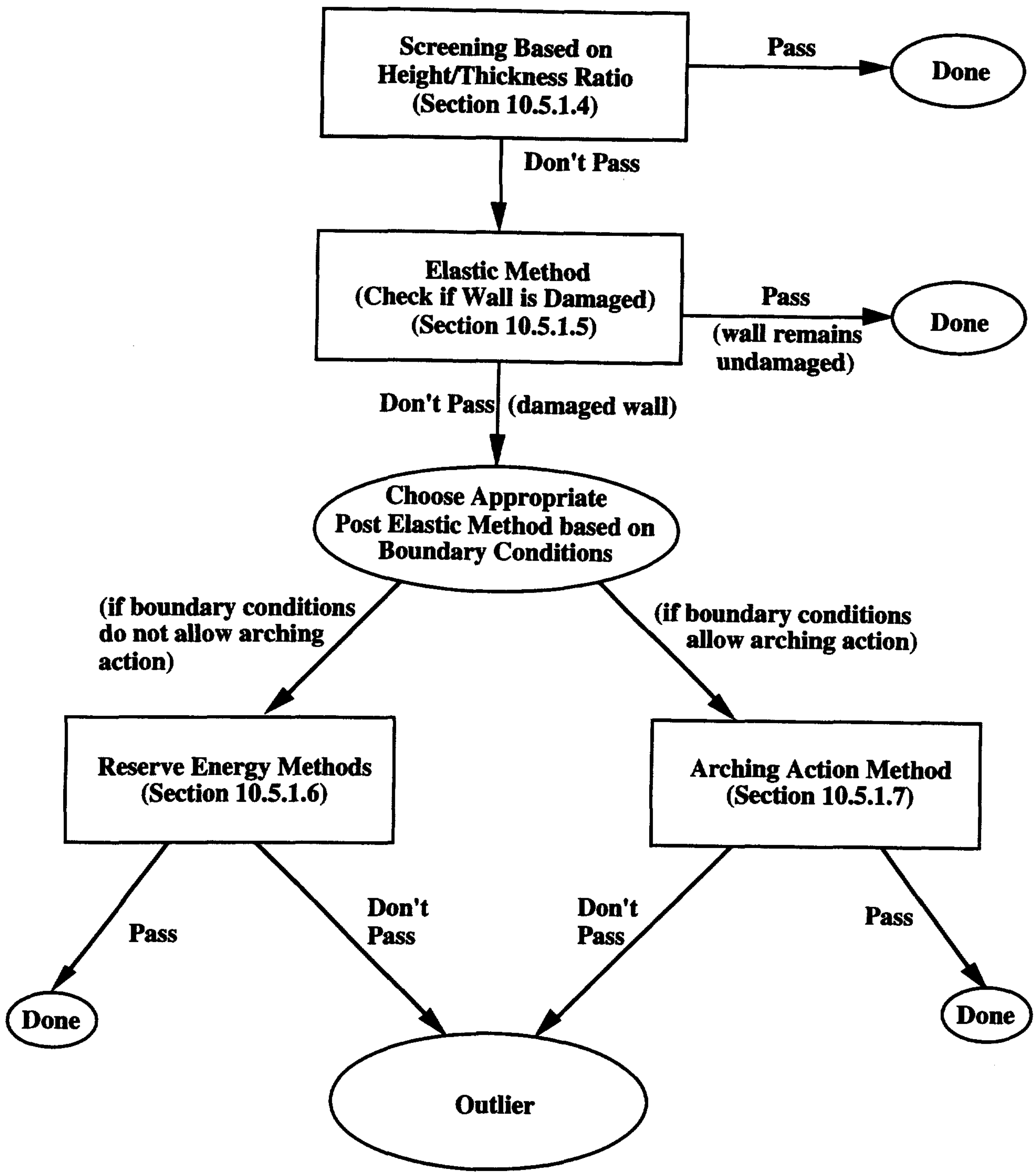
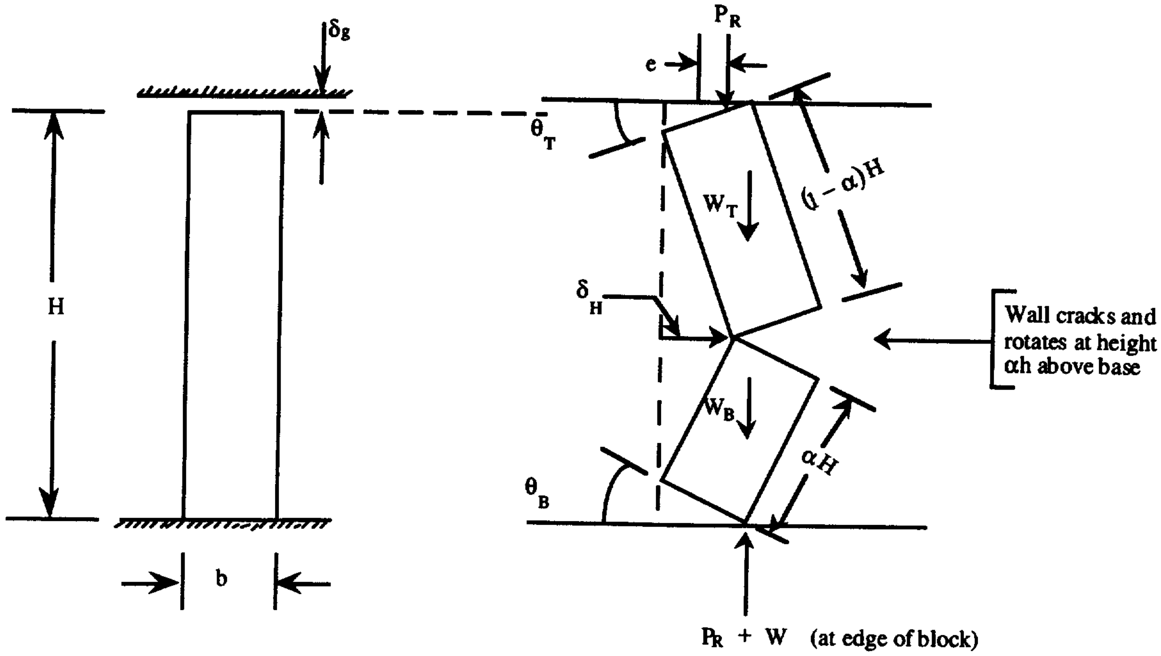
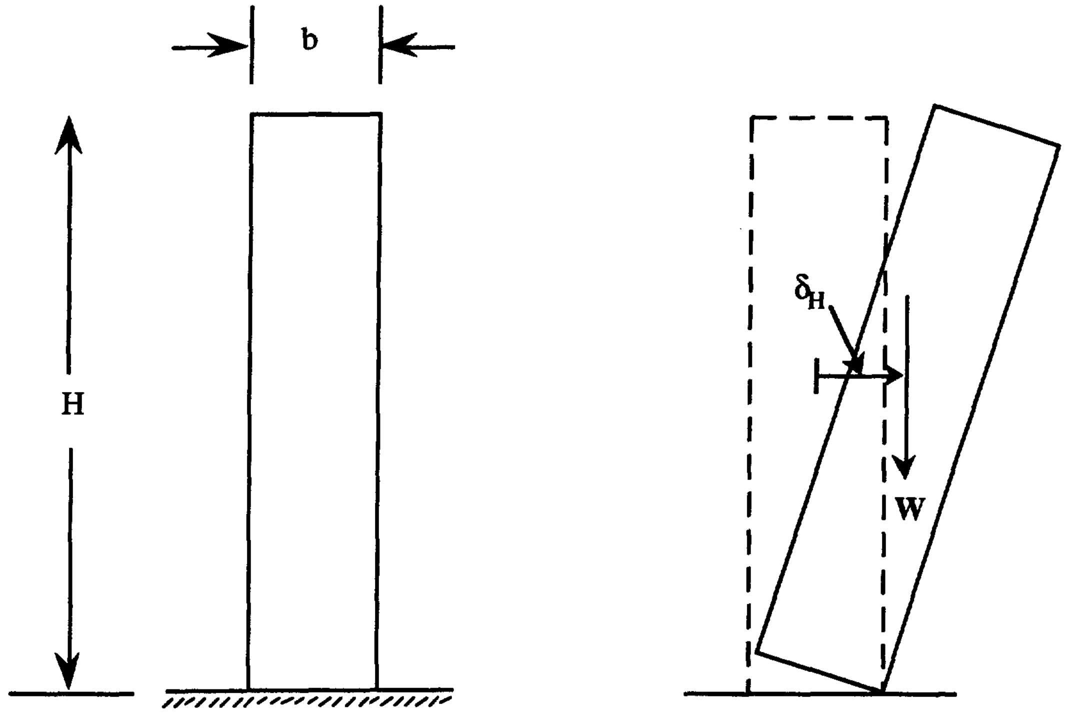


Figure 10.5.1-1 Methods for Evaluation of Out-of-Plane Bending of Non-Bearing Infill or Partition Unreinforced Masonry Walls in Section 10.5.1



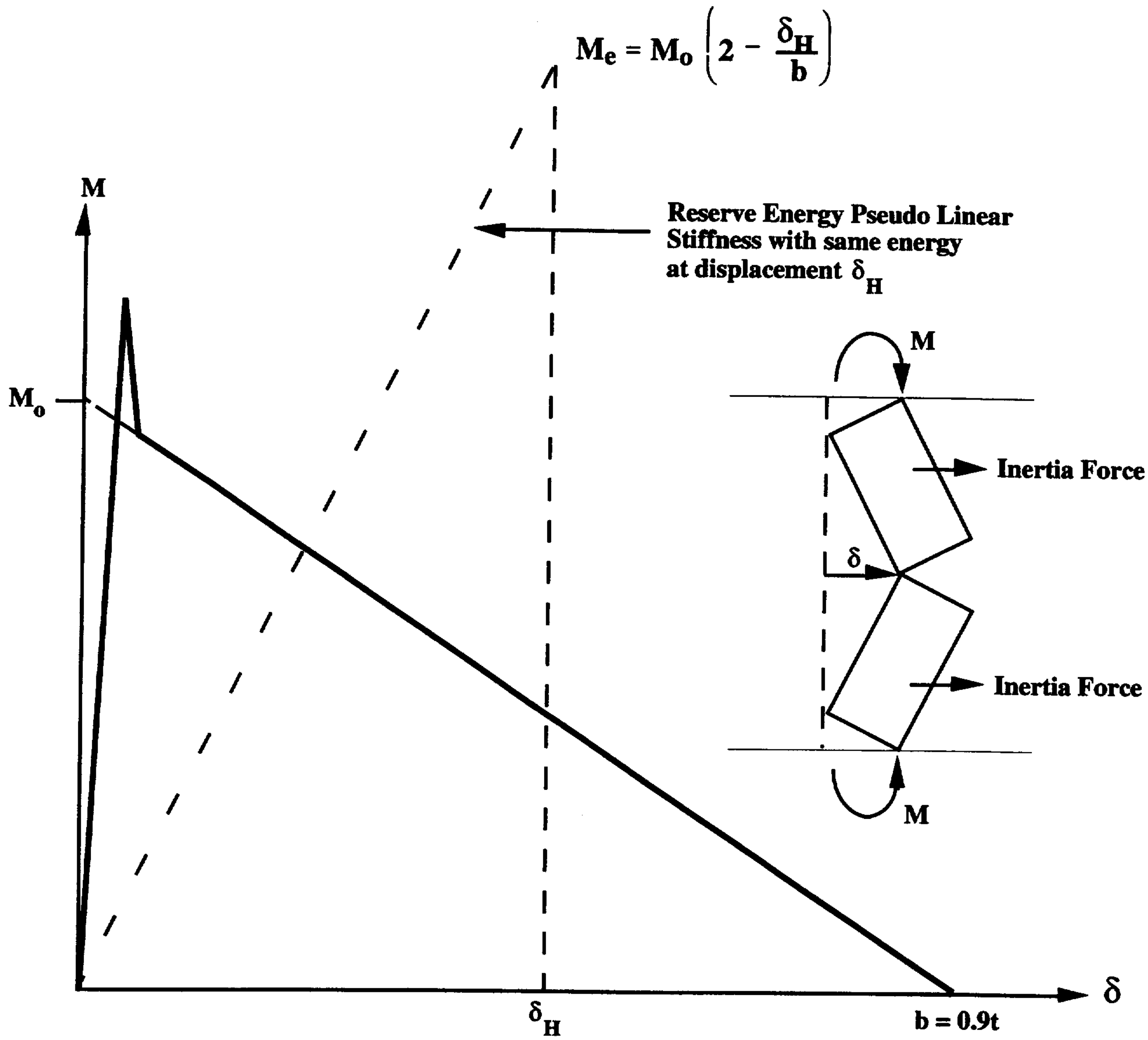
P_R	=	in-plane compressive force zero for Reserve Energy Method (non load bearing wall) increases with displacement for Arching Action Method
W_B	=	$W\alpha$
W_T	=	$W(1-\alpha)$
W	=	block wall weight
α	=	parameter which locates crack location
e	=	load eccentricity from centerline of wall
H	=	wall height
b	=	effective wall thickness (= 0.9 actual wall thickness)
δ_H	=	lateral displacement
δ_g	=	gap between wall and upper support
θ_B	=	angle of rotation of bottom block
θ_T	=	angle of rotation of top block

Figure 10.5.1-2 Wall Properties for Reserve Energy and Arching Action Methods



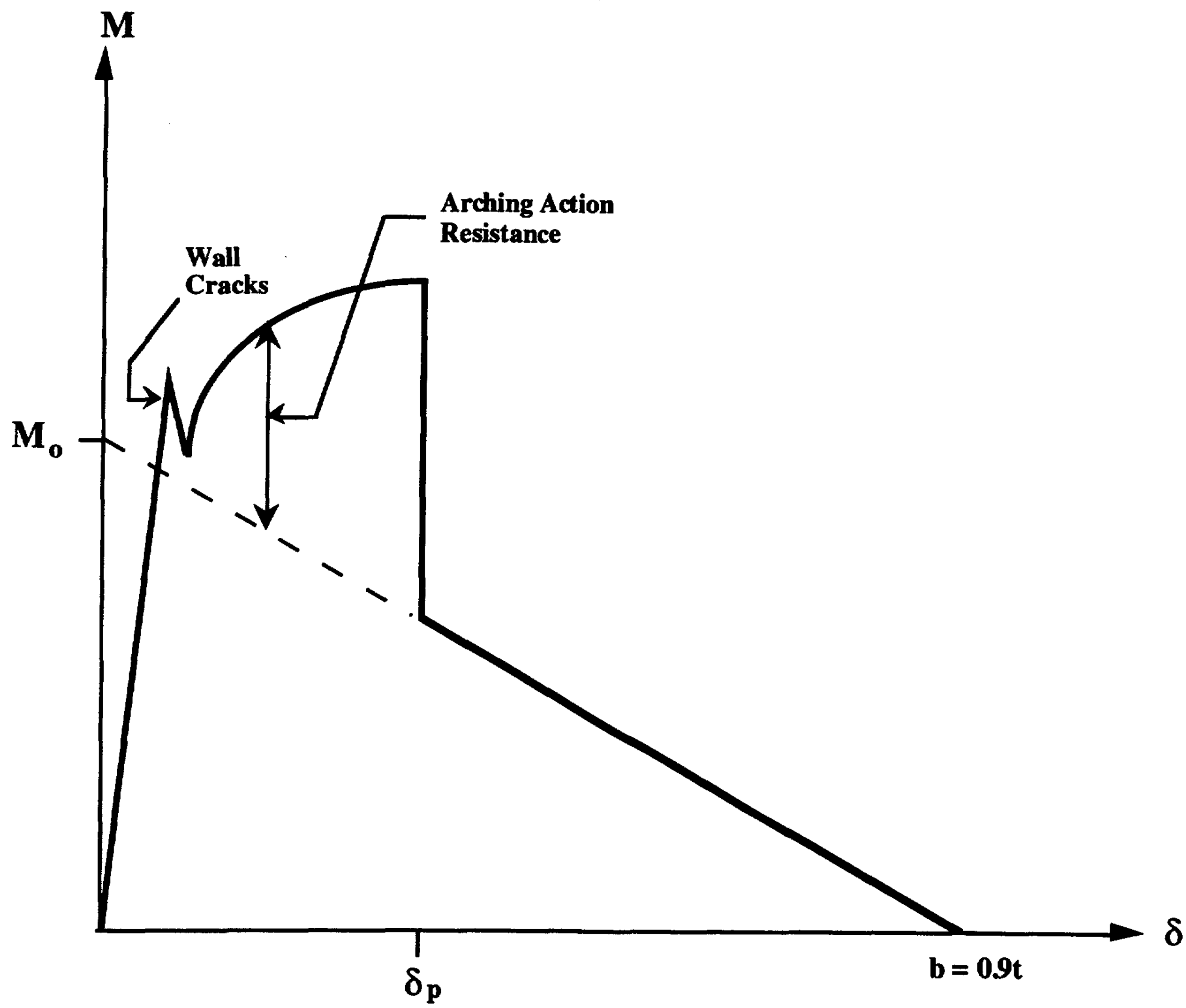
W	=	Block wall weight
H	=	wall height
b	=	effective wall thickness (= 0.9 actual wall thickness)
δ_H	=	Lateral displacement

Figure 10.5.1-3 Properties for a Cantilever Wall for Reserve Energy Method (Large gap at top of wall, non load bearing, and no lateral restraint at top of wall)



M	=	restoring moment
M_o	=	actual moment at zero displacement
M_e	=	effective moment
b	=	effective wall thickness
t	=	actual wall thickness
δ_1 δ_H	=	out-of-plane displacements

Figure 10.5.1-4 Restoring Force for Reserve Energy Method



M	=	restoring moment
M_o	=	actual moment at zero displacement
b	=	effective wall thickness
t	=	actual wall thickness
δ_p	=	out-of-plane displacement at which ultimate capacity is reached
δ	=	out-of-plane displacement

Figure 10.5.1-5 Restoring Force for Arching Action Method

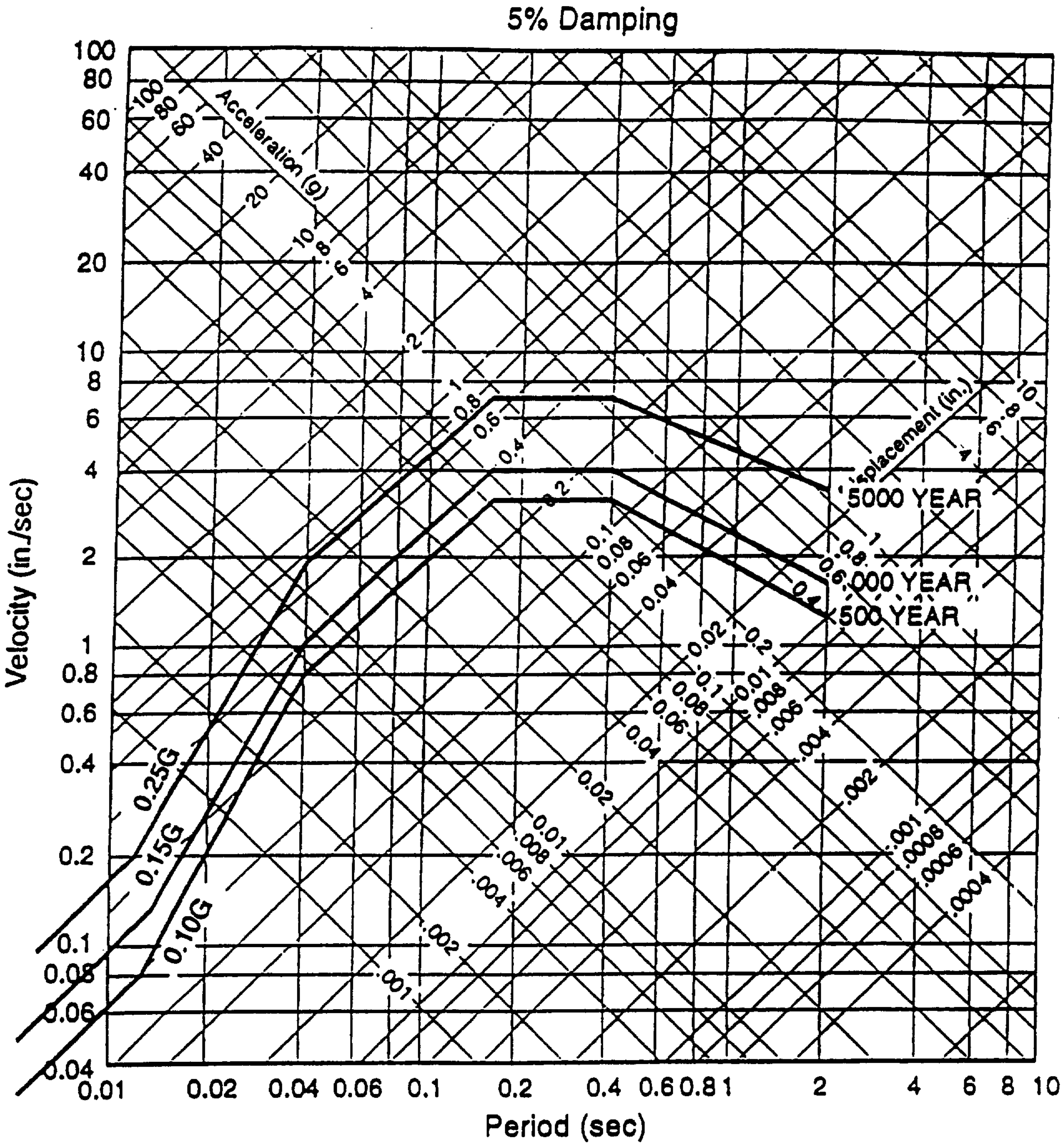


Figure 10.5.1-6A Portsmouth-Uniform Hazard Response Spectra for Horizontal Ground Motion

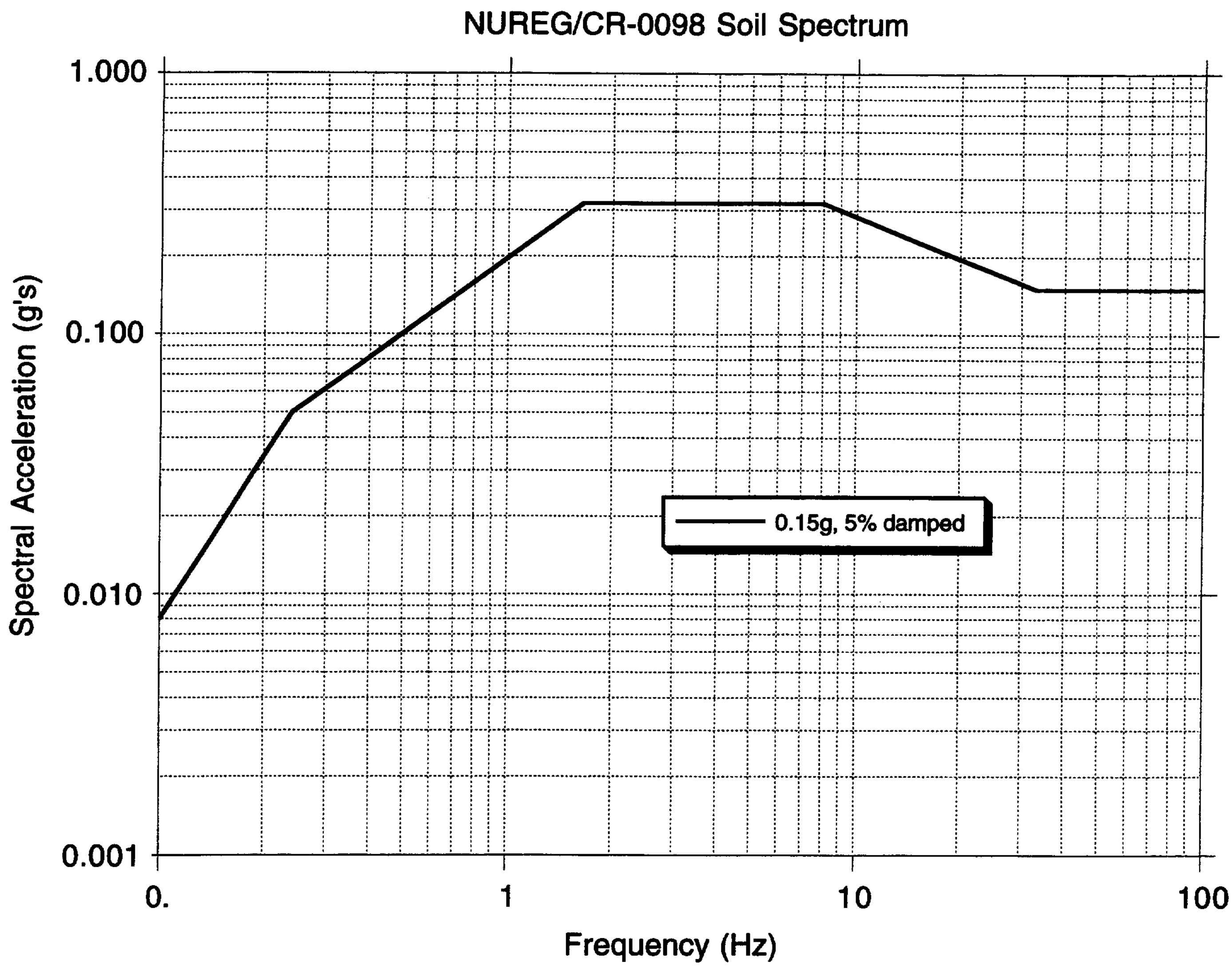


Figure 10.5.1-6B NUREG/CR-0098 Median Soil Spectrum (Ref. 72)

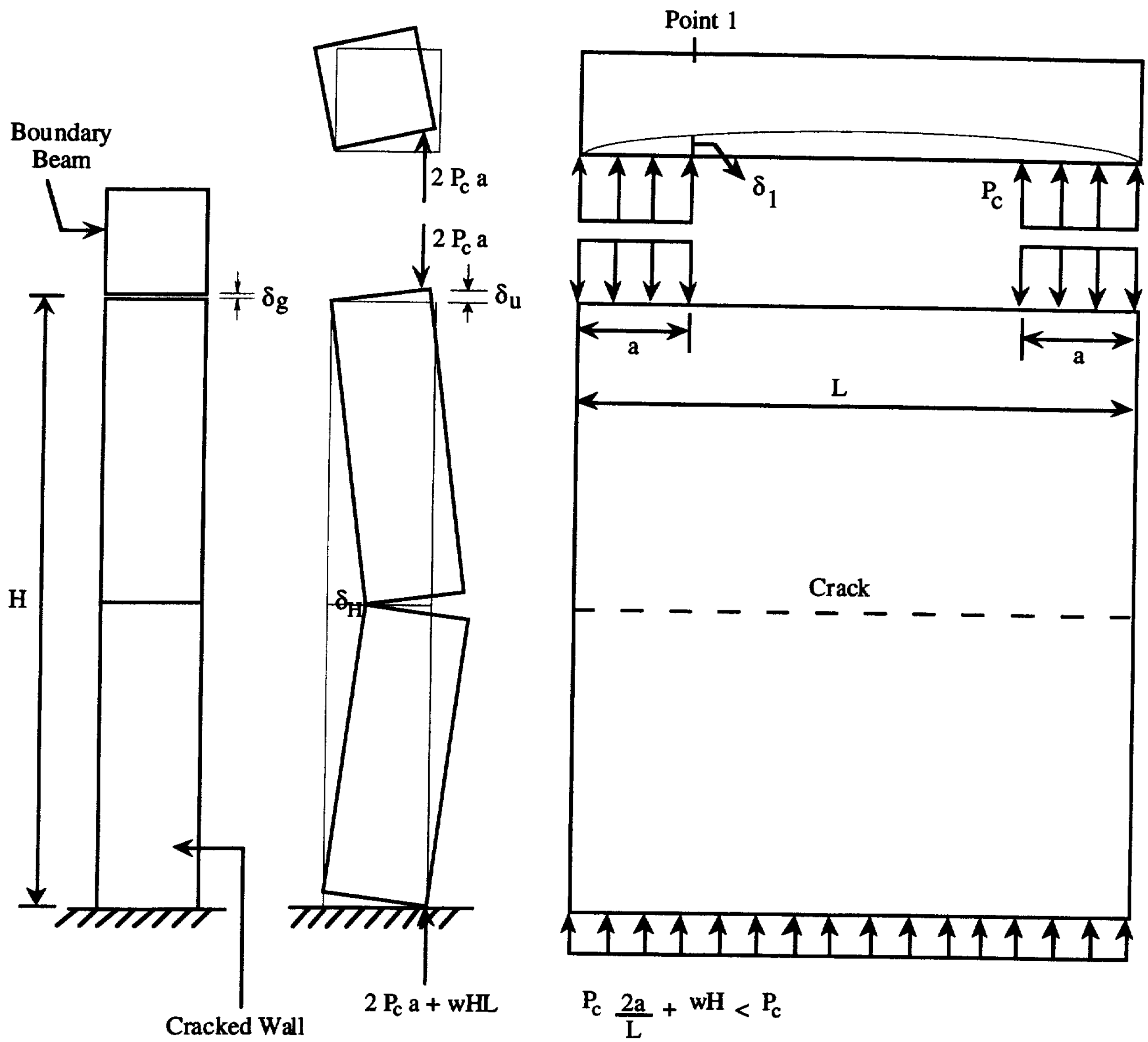
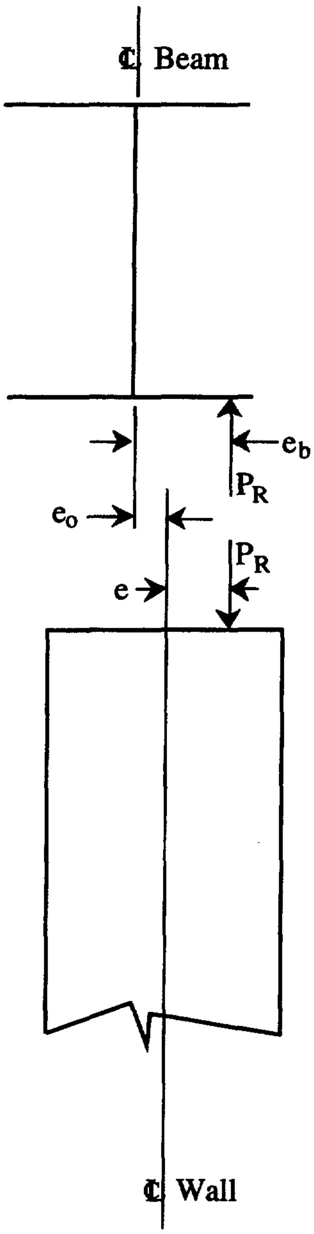


Figure 10.5.1-7 Arching Kinematics and Assumed Load Distribution along Length of Top Beam



e_o = offset between beam centerline and wall centerline

e = eccentricity to load P_R from wall centerline

e_b = eccentricity to load P_R from beam centerline

If beam twists more freely than top of wall rotates (typical for steel beam)

take $e_b = o$
$e = -e_o$

If twisting stiffness of beam is sufficiently large, then the beam twists less than the top of the wall rotates (typical for concrete beam)

take $e \approx 0.45\ b_f - e_o \leq 0.45t$
$e_b = e + e_o$
b_f = flange width of beam

Figure 10.5.1-8 Geometry of Beam, Wall, and Confining Force

10.5.2 RAISED FLOORS

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of raised floors which are included in the Seismic Equipment List (SEL). The guidelines contained in this section are based on Section 4.4 of "Practical Equipment Seismic Upgrade and Strengthening Guidelines" (Ref. 60), Chapter 6 of "Data Processing Facilities: Guidelines for Earthquake Hazard Mitigation" (Ref. 121), and Chapter 9c of the "Seismic Safety Manual" (Ref. 32). In Chapter 6 of Reference 121, further detailed information on the seismic performance of raised floors and techniques for upgrading their seismic capacity is contained in the following sections: Descriptions of some of the more common floor systems and their strengths and weaknesses under earthquake loading; Specific guidelines for the seismic design, analysis, testing, and inspection of new raised floor systems; and Guidelines for analysis, retrofit design, and testing of existing raised access floors. Guidelines in this section of the DOE Seismic Evaluation Procedure cover those features of raised floors which experience has shown can be vulnerable to seismic loadings.

Because of extensive cabling requirements, components in computer facilities, data processing facilities, and control rooms are often supported on a raised floor with removable panels that may or may not be supported by stringers. A typical raise floor system is shown in Figure 10.5.2-1. A raised floor system forms the basic foundation or support for computer and data processing equipment, creates a space for a HVAC air plenum, and provides a protective shield for subfloor utilities vital to the operation of the equipment. The equipment supported on raised floors often costs hundreds of times more than the cost of the floor. Because of the cost of the equipment on a raised floor, earthquake-induced damage to the floor has a very high property loss potential. Furthermore, reconstruction of the collapsed floor and reinstallation of subfloor power, cooling, and signal cables could take a considerable amount of time. Potential damage evidenced in raised floor systems include buckling of support pedestals, buckling of floor panels, misalignment of floor penetrations, shifting of the entire floor system, and tipping of equipment supported by the floor.

For raised floor systems, the following seismic parameters should be evaluated:

- Seismic Demand Spectrum (SDS) at location of floor anchorage (see Section 5.2)
- dynamic stability or ability to withstand tipping and buckling capacity of pedestals
- type of anchorage system (leveling pads, skids, adhesives, clips, bolts, none)
- load path to load-bearing floor or foundation
- geometry and size (aspect ratio, height, width, length)
- relative strength and stiffness (stiff, flexible, strong, medium, weak)
- spacing of pedestals
- penetrations in the raised floor system
- operational considerations (weight being supported by floor, distribution of weight)

Large computer or control room raised floors may be susceptible to earthquake-induced damage due to tipping of the support pedestals. Figures 10.5.2-2 and 10.5.2-3 show examples of support pedestals that are typically slender, relatively long, and unanchored to the load-bearing floor or foundation. In addition, many raised floor systems lack lateral bracing between the pedestals (see Figure 10.5.2-4) which would provide horizontal stiffness.

To resist potential earthquake-induced damage, raised floor systems should be properly anchored by drilling holes in the base plates of supporting pedestals and installing anchor bolts. The anchor bolts can be evaluated using the procedures in Chapter 6. Many raised floor systems use an adhesive to attach the pedestals to the load-bearing floor or foundation. Test results have indicated that this adhesive is not adequate for withstanding significant lateral motion.

Earthquake and test experience has indicated that the unbraced pedestals and the weld to the pedestal base plate are often too weak to transfer the required lateral loads. Bracing schemes as shown in Figures 10.5.2-5 should be provided to create moment-frame action of the raised floor systems, to increase the lateral stiffness of the raised floor system, and to avoid concerns about the weld to the pedestal base plate. Potential flexibility of the threaded screw connections and weak welds, such as tack welds, to the pedestal should be evaluated.

In addition to strengthening the raised floor support system, the penetrations in the floor systems should be carefully evaluated. In many cases, the equipment on the raised floor is not anchored so there needs to be adequate accommodations for movement of the equipment during an earthquake. If there are extensive floor penetrations, the equipment on the raised floor may roll into, tip on, or catch on the penetrations. This action may cause a large concentrated lateral overload on the floor system as well as cause local floor breakup due to panel buckling. The floor penetrations should be modified to prevent equipment entry or covered with special air vents that permit the equipment to traverse the floor without penetration. Special precautions may be required to anchor the equipment through the raised floor or tether it to prevent it from catching in the penetrations. For light equipment on a braced floor, connecting to the bracing at the stringers may be adequate restraint. The use of tethers is discussed below.

Strengthening of the raised floor will not necessarily provide a system capable of resisting the lateral loads associated with heavy computer or control equipment. Separate anchorage for these items of equipment should be provided. The most desirable strategy for upgrading the seismic capacity of computer equipment typically involves either floor anchorage, vertical bracing schemes, or the use of tethers. The anchorage of the equipment on the raised floor may be used for the following conditions:

- the equipment is relatively heavy
- analysis of the equipment indicates that it will tip
- the equipment is closely spaced and will impact
- the internal components have low vulnerability to vibratory motion
- the cabinet frame has sufficient strength and stiffness to support the equipment without supplemental bracing.

Because unbraced raised floors cannot carry significant lateral loads, independent anchorage and support for equipment meeting one or more of the conditions listed above should be to a load-bearing floor or foundation. With the independent support, the raised floor should not be part of the load path for the anchorage of large computer and control equipment. The base of the equipment should be evaluated to determine if it has adequate capacity to support the anchorage loads.

An approach for independently securing equipment on top of a raised floor is to use under-floor cable tethers which allow for limited movement of the equipment. The cable tethers secure the equipment by providing a support path between a floor or load-bearing wall and the base of the equipment. As discussed in Reference 32, the following factors should be considered when using a tethering system:

- openings in the raised floor should have raised edges or curbs to prevent the base of the equipment from sliding into the opening
- the equipment should be stable against overturning when an appropriate coefficient of friction (judgment is required) is assumed between the raised floor and the base of the equipment
- there should be sufficient space between equipment to prevent seismic interactions
- elastomeric pads or bumpers may be used between closely spaced equipment
- the location of tether anchors and cable attachments to the equipment should consider the distribution of mass and stiffness within the equipment
- the design of the tether anchorage should consider the interaction with the raised floor if the cable becomes taut
- attached lines to the equipment should have sufficient slack to accommodate the constrained movement of the equipment

A second approach for independently anchoring computer equipment to a load-bearing floor or foundation is to use a separate support system, such as a diagonally - braced frame, for the equipment. This support system must be adequately anchored, have adequate lateral bracing, and have an appropriate load path from the equipment to the support system. If the equipment anchorage to the separate support system passes through an unbraced raised floor, interactions between the floor and the equipment anchorage should be considered.

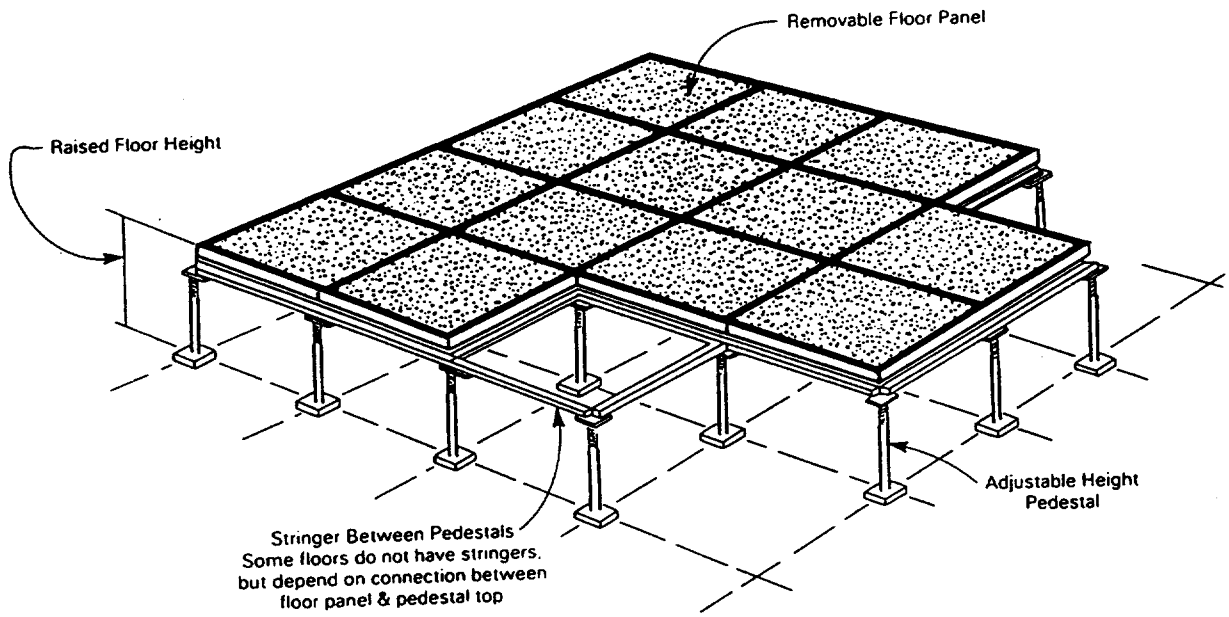


Figure 10.5.2-1 Raised Floor System (Figure 6.1 of Reference 121)

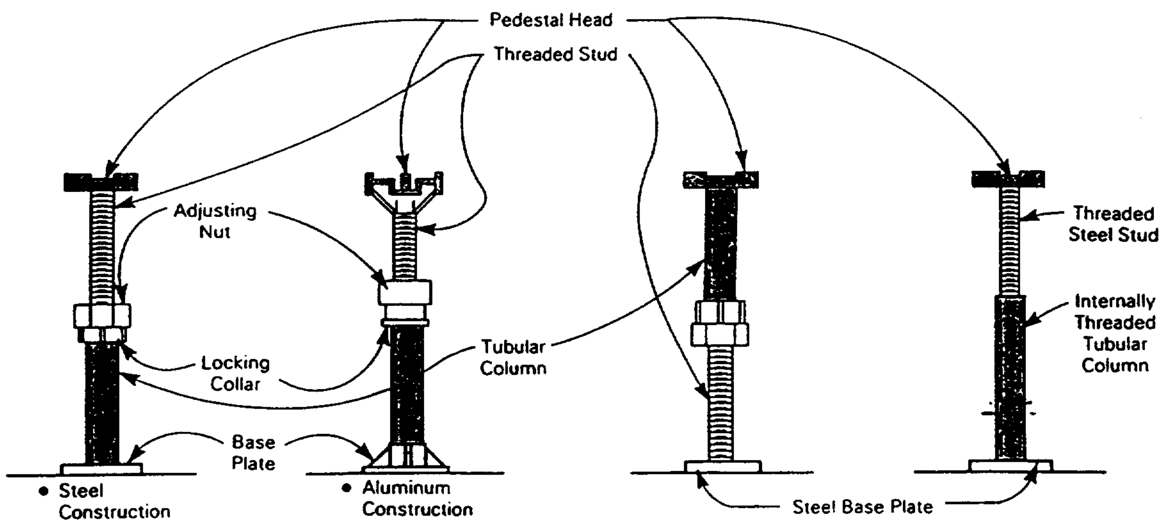


Figure 10.5.2-2 Pedestal Types (Figure 6.2 of Reference 121)

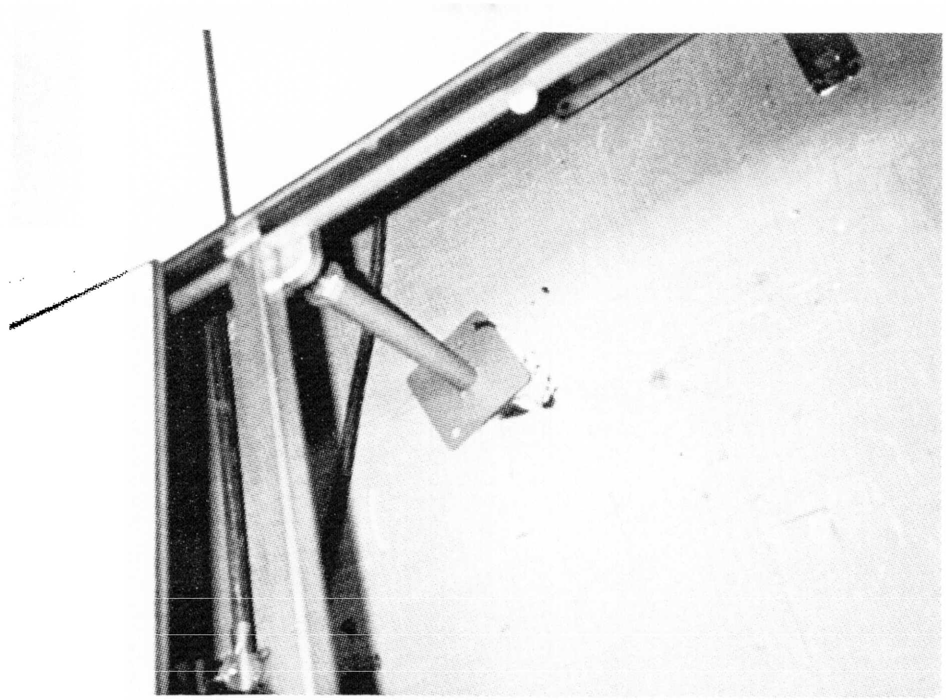


Figure 10.5.2-3 Raised Computer Floor Supported by Pedestal and Leveling Screw (Figure 4-30 of Reference 60)

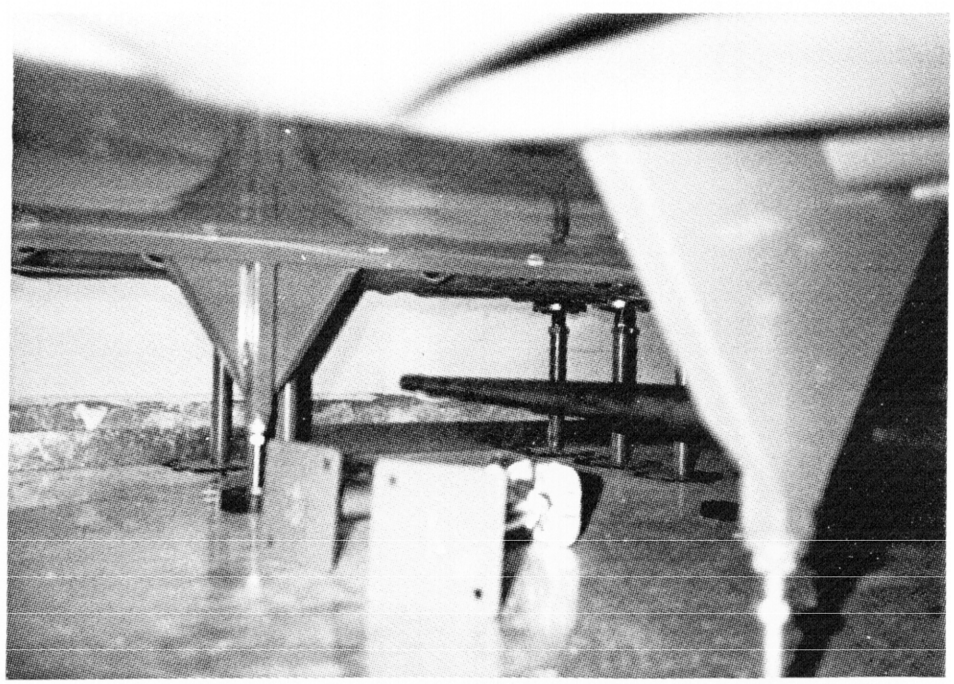


Figure 10.5.2-4 Raised Computer Floor Showing Lack of Lateral Bracing (Figure 4-31 of Reference 60)

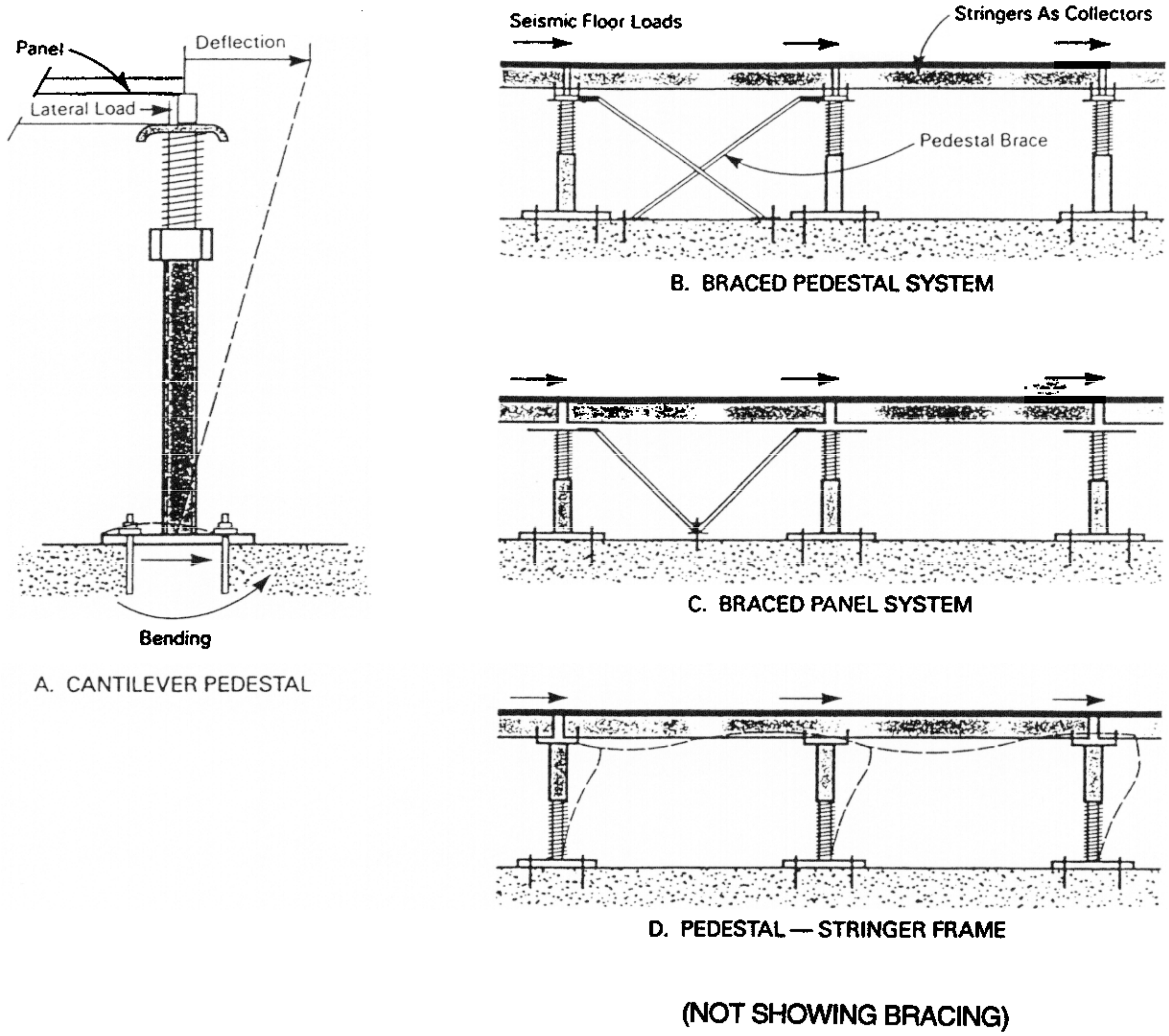


Figure 10.5.2-5 Lateral Force Resisting Systems (Figure 6.5 of Reference 121)

10.5.3 STORAGE RACKS

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of storage racks which are included in the Seismic Equipment List (SEL). The guidelines contained in this section are based on Sections 4.6.5 and 4.8 of "Practical Equipment Seismic Upgrade and Strengthening Guidelines" (Ref. 60). Guidelines in this section cover those features of storage racks which experience has shown can be vulnerable to seismic loadings.

Raw materials and finished products are typically stored on racks, in bins, or in stacks. Storage racks range from light metal shelving (see Figure 10.5.3-1) to heavy industrial grade shelving (see Figure 10.5.3-2). Inventory is extremely susceptible to earthquake-induced damage if racks or bins have no identifiable lateral load carrying system (see Figure 10.5.3-3). During an earthquake, items may slide off shelves or shelving may collapse which causes the contents to spill to the floor. If hazardous chemicals are involved, the resulting toxic chemical spill can be extremely dangerous and expensive to clean up.

The seismic evaluation of storage racks should emphasize the following considerations:

- anchorage
- structural capacity
- lateral bracing
- load path
- connection details
- restraints for contents

The structural capacity of a storage rack should be evaluated, especially its capacity for lateral loads. It may be difficult to determine the capacity of the rack without performing some calculations to determine member strengths and the modal, or stiffness, characteristics of the frame. Judgment may be required for determining the appropriate model for the connection details in a rack system. The connections in rack systems range from welded connections to slip joints. According to the provisions of Section 5.4, the capacity of the rack should be compared to the Seismic Demand Spectrum (SRS) at the anchorage location of the rack.

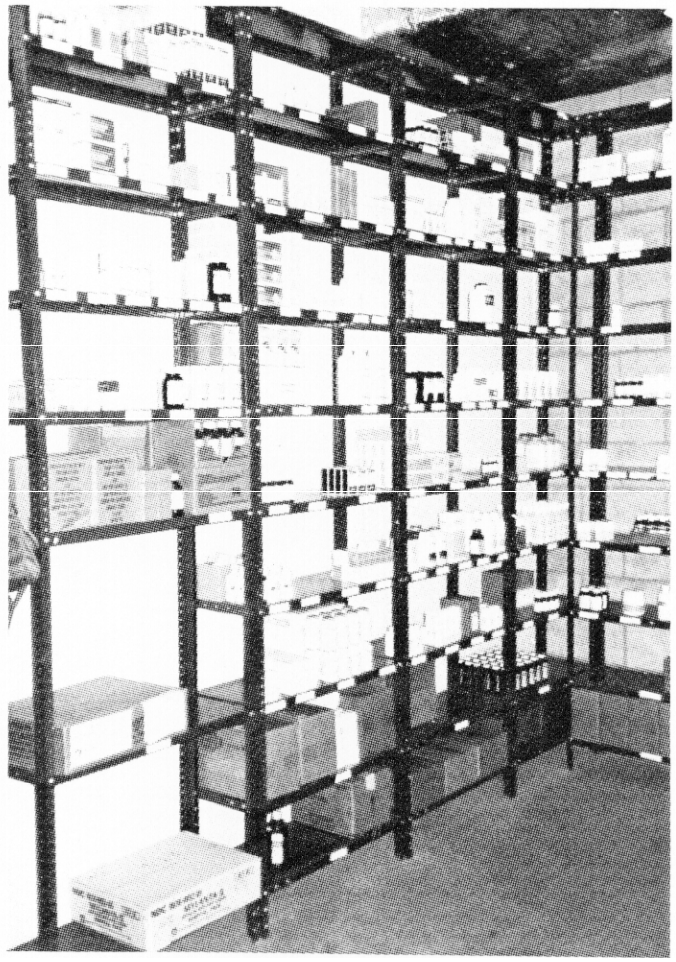
Storage racks should be evaluated to determine if they have adequate anchorage and if lateral bracing is present and of sufficient size to accommodate seismic loads. Tall racks should be anchored to walls with adequate capacity, the floor, and/or each other to prevent overturning. Most rack units have holes provided in their base plates and legs to accommodate anchor bolts. The screening evaluation for anchor bolts is provided in Chapter 6. The capacity of the floor to resist the anchorage loads should be evaluated. Many rack systems are leveled with shims and the excessive use of shims may reduce the capacity of the anchorage for those systems. If the rack is anchored to an unreinforced masonry (URM) wall, the capacity of the wall should be evaluated according to the provisions of Section 10.5.1 including the lateral loads of the racks.

Since racks are relatively flexible, extensive use of lateral bracing is useful in increasing the seismic capacity of the rack and in limiting earthquake-induced damage. Bracing should be provided at the ends and along the back side as shown in Figure 10.5.3-4. In addition to bracing, the load path in the structure should be evaluated. The bracing should attach to the structural members of the rack and these members should have sufficient capacity to withstand the earthquake-induced lateral demand. Many racks are designed only for vertical loads, so the effects of lateral loads should be evaluated. Additional information on the seismic design of storage racks is available from the Rack Manufacturer's Institute. Finally, possible reductions in the structural capacity of a storage rack may result from improper assembly of the rack or damage from operational use, such as forklift damage. Manufacturer's data should be used to determine if the rack was properly assembled and is being used as designed.

Horizontal shelves that are structurally attached to the supporting frame members are preferred as part of the connection details in a storage rack. If the rack has removable shelves, these shelves cannot be considered part of the lateral force resisting system. Loose pieces of wood spanning between frames may fall during an earthquake and should be restrained. Heavier stock should be moved to lower shelves to prevent injury to personnel and to minimize damage. Whenever possible, restraint should be provided for equipment or stock that can slide off during earthquake motions. Methods of achieving restrain include installation of a steel angle (lip) at the front edge of each shelf or an elastic band or tensioned wire across the opening. If feasible, removable restraints can also be provided across the front of the rack to preclude materials from sliding off shelves as shown in Figure 10.5.3-4.

During an earthquake, the support structure for drums supported on a rack may collapse if it does not have adequate lateral bracing and seismic anchorage. Poorly restrained canisters and drums may fall and/or roll causing them to possibly spill their contents, to damage other equipment, and injure personnel. Methods of restraining them include providing positive anchorage to the floor or a wall with adequate capacity, storing them in well-braced and anchored racks, or storing them horizontally on the floor.

Storage bins are temporary storage containers stacked on top of each other. Bins are often stacked very high with no lateral supports. In a strong earthquake, the upper bins can fall causing damage to contents and pose a possible life safety hazard. Materials stored in bins or stacks should be assessed to determine their stability under earthquake loads. Often, the seismic requirements of these components is in direct conflict with operational requirements. However, if materials are extremely hazardous or are expensive to replace, mitigation measures should be considered to provide positive restraint. These measures might include the installation of permanent racks, minimizing stack heights to 2 or 3 layers in height, or restraining existing stacks through tiedowns.



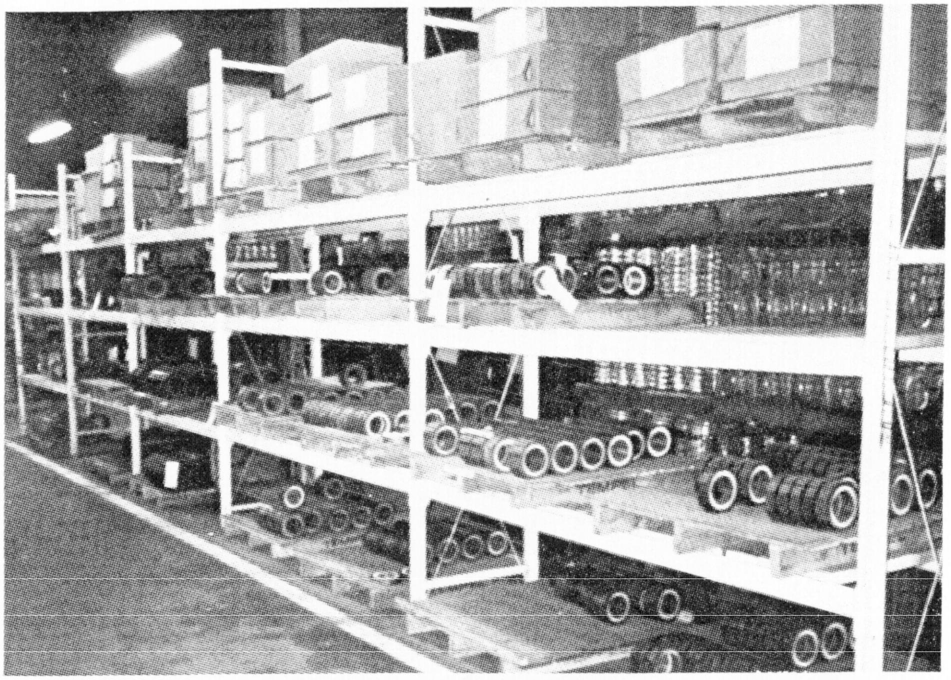


Figure 10.5.3-2 Unanchored Industrial Grade Shelving (Figure 4-63 of Reference 60)

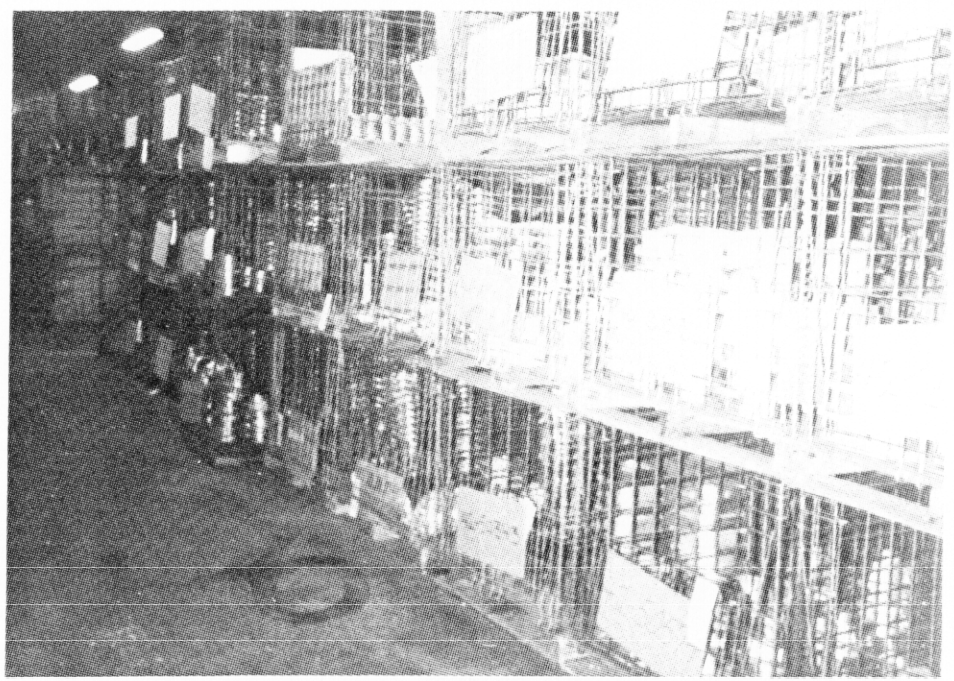


Figure 10.5.3-3 Unanchored Storage Bins (Figure 4-64 of Reference 60)

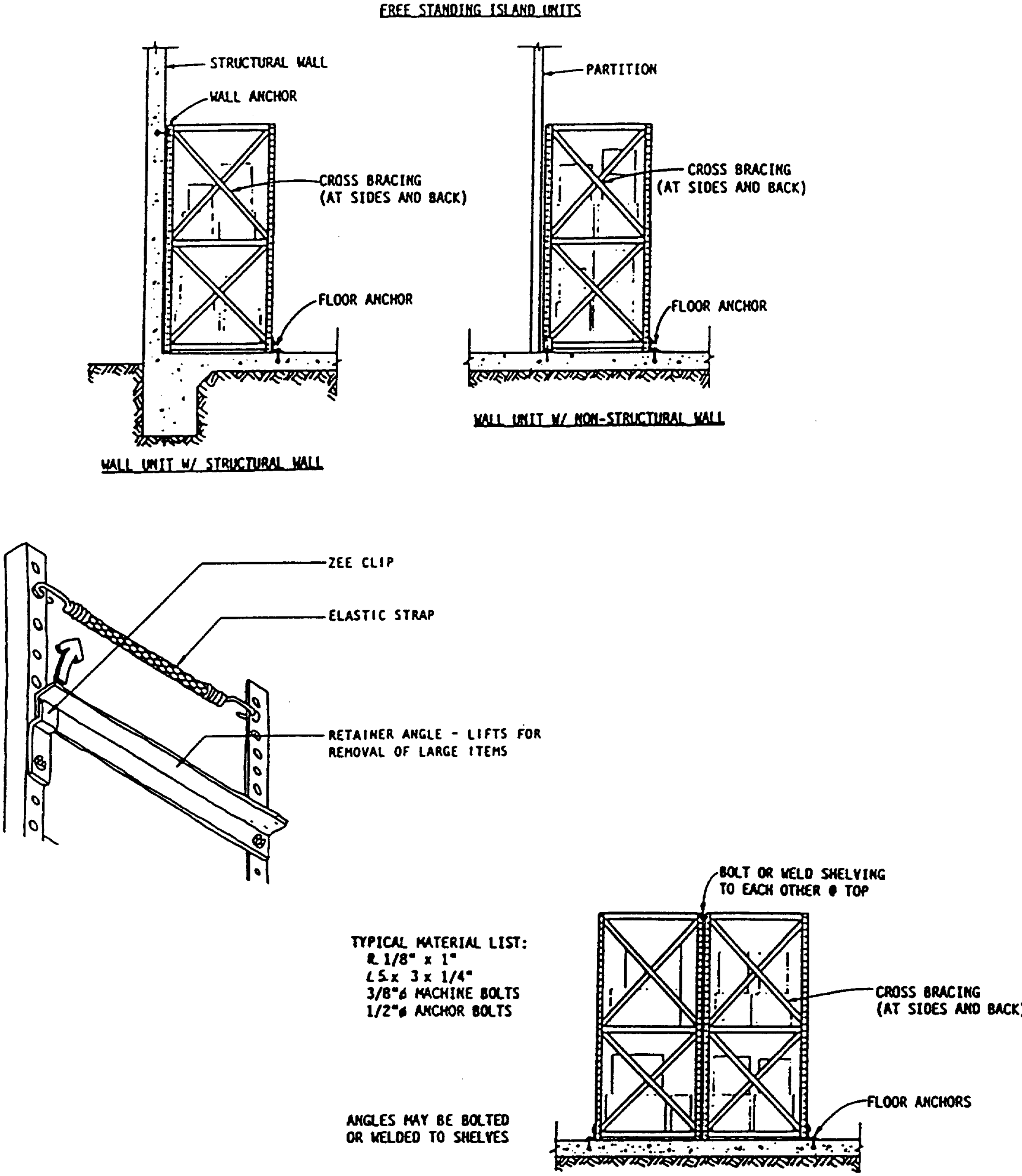


Figure 10.5.3-4 Approaches for Anchoring Storage Racks (Figure 4-67 of Reference 60)